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Proceedings of the American Society of Civil Engineers

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December, 1958

Papers

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HYDRAULICS DIVISION
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METEOROLOGICAL ASPECTS OF STORM SURGE GENERATION[a]

D. Lee Harris[1]
(Proc. Paper 1859)

INTRODUCTION

The coastal floods, produced by Hurricane AUDREY in June 1957 claimed ·e than 300(1) lives and destroyed more than 100 homes. Even in this en- tened year of 1957, when hurricane warnings are broadcast by radio and vision long before the storm arrives, the high tides produced by the storm :inue to claim more lives than all other aspects of hurricane behavior. Al- ıgh many of the newcomers in the Cameron area evacuated the threatened .on soon after the hurricane warnings were announced, many of the old- ers remained because they had weathered previous hurricanes with little .culty and they did not expect this one to be more severe than the others. w facts concerning hurricanes and their effects on sea level, which if fully erstood, may prevent future disasters of this kind, are presented in this er.

This paper is concerned primarily with the meteorological aspects of m surge generation by hurricanes. The physical aspects of hurricane m surge behavior along the Atlantic and Gulf coasts of the United States been discussed in greater detail in Harris.(2) Zetler(3) gave an extensive tment of the effects of hurricanes on sea level at Charleston, S. C. field and Miller(4) considered the effects of several recent Atlantic Coast ·icanes. Hubert and Clark(5) published charts showing the high water ks established after 16 Atlantic and Gulf coast hurricanes. Reid(6) de- ped a storm tide index based on the size and intensity of the hurricane and slope of the continental shelf.

: Discussion open until May 1, 1959. To extend the closing date one month, a ritten request must be filed with the Executive Secretary, ASCE. Paper 1859 is art of the copyrighted Journal of the Hydraulics Division, Proceedings of the merican Society of Civil Engineers, Vol. 84, No. HY 7, December, 1958.

Presented at the ASCE Hydraulics Conference in Cambridge, Mass., ugust, 1957.

Office of Meteorological Research, Storm Surge Research Unit, U. S. Veather Bureau, Washington, D. C.

Hurricane Models

The wind and pressure distributions of a hurricane are never known with desired accuracy before the storm strikes land. Hence, in forecasting stor surges and in the design of engineering works to be constructed as protecti from some future hurricane, it is necessary to use a storm model. Such models can be constructed in a variety of ways depending on the use for wh they are intended. For storm surge work it is desired to describe as accu-rately as possible the distribution of atmospheric pressure and sustained w speeds near the surface of the sea.

The pressure in many hurricanes can be represented with a close appro mation by an equation of the type

$$P = P_o + (P_n - P_o) F (r/R),$$

where P_n = the pressure just outside the storm,

P_o = the pressure at the center of the storm,

R = some characteristic radius of the storm,

r = distance from the center of the storm,

$F(r/R)$ indicates a function of r/R.

The radius of the eye, the distance from the center to the zone of maxim winds, and the distance from the center to the outermost closed isobar have all been suggested for use as the characteristic radius. The proper choice probably depends on whether it is desired to obtain the maximum accuracy the high wind speed zone or for the entire storm. $(P_n - P_o)$ is intended as measure of the intensity of the storm, but P_n is likewise difficult to define explicitly. Some workers prefer to use the pressure of the outermost clos isobar, still others prefer some other definition. Virtually all workers in this field use the best obtainable estimate of the lowest pressure in the sto for P_o. Because an objective definition of P_o is easily constructed, many writers use this value to estimate the intensity of the storm.

The first approximation of the wind speed within any cyclone is given by the gradient wind equation. This can be expressed in terms similar to Eq. by:

$$V_g^2/r + fV_g = \rho^{-1}(P_n - P_o) F'(r/R).$$

V_g = gradient wind speed

f = coriolis parameter

ρ = density of air

F' = derivative of F with respect to (r/R)

Near the center of a hurricane V_g^2/r is considerably larger than fV_g; h the second term on the left can be neglected within the accuracy of this ap-proximation. Thus the gradient wind, and presumably, the true surface wi also, is approximately proportional to the square root of $(P_n - P_o)$. This r lationship is often used in estimating the maximum wind speed in a hurricane.(11)

A hurricane model developed along the lines indicated by Eqs. (1) and (2) is described by the following equations:

$$\frac{P - P_o}{P_n - P_o} = e^{-R/r} \tag{3}$$

$$V_g^2/r + fV_g = \frac{1}{\rho}(P_n - P_o) \quad e^{-R/r} \tag{4}$$

The symbol R in this model has been identified with the radius of maximum wind speed. The agreement between this model and the observations of an exceptionally well observed hurricane is shown in Fig. 1. The insert map gives the storm track. The dots on the left give the observed pressure at several stations in the vicinity of Lake Okeechobee, Florida. The solid line gives the theoretical pressure profile fitted to three points within 50 miles of the storm center. The corresponding theoretical wind profile is given by the upper curve on the right hand side of the figure. The observed winds at one station are given by the dots below this curve. A solid line has been drawn through these dots by eye in order to obtain a smooth profile. The observed wind speed varies in a systematic way from about 65% of the computed wind speed at the outer edge of the data shown to almost 90% of the predicted value near the zone of maximum wind speed. Equally good agreement between the theoretical and observed wind speeds has been obtained in only a few storms. This lack of agreement between the theoretical and observed winds is due in part to the elementary nature of the model, but perhaps equally to the lack of first class wind records near the center of hurricanes. The parameters for this model have been computed for a large number of storms and were first published in Hydrometeorological Report No. 32.(7)

Data for new storms are added and data for old storms are corrected as new information becomes available. An up-to-date (August 1, 1957) version of these data is included in this paper in Table I.

Eqs. (2) and (4) can be valid only for stationary storms. The wind speeds on the right hand side of a moving storm are generally higher than those observed to the left of the storm track. A simple but incomplete explanation, widely accepted for more than half a century, is based on the assumption that the forward speed of the storm must be added to the rotary speed on the right hand side and subtracted on the left. This would give an asymmetry in the wind velocity on the right and left sides of the storm track due to the forward speed of the storm. This assumption usually leads to corrections in the right direction but not of the proper magnitude. Winds of hurricane intensity are significantly affected by friction with the underlying water or land surface. The effects of friction and of the inflow necessary to feed the vertical currents, prominent throughout the region of hurricane winds, may exceed the effects of the forward motion of the storm, at least near the surface.

Although Eq. (4) is too simple to permit an adequate determination of the wind field in a hurricane, it is possible to make use of empirically derived relations between the wind speed at nearby locations over water and over land, and between the observed wind speed over various types of frictional surfaces and the theoretical wind speed to determine a usefully accurate picture of the wind field. Figs. 2a and b show the reconstructed pressure and wind fields for the New England Hurricane of September 21, 1938.(8) In the derivation of

DATE	P_o in.	P_n in.	P_n-P_o in.	P_o mb.	P_n mb.	P_n-P_o mb.
Aug. 28, 1893	28.28	29.61	1.33	958	1003	45
Oct. 1, 1893	28.22	29.99	1.77	956	1016	60
Oct. 13, 1893	28.33	29.61	1.28	959	1003	44
Oct. 2, 1898	28.82	30.03	1.21	976	1017	41
Oct. 31, 1899	28.70	30.49	1.79	972	1033	61
Sept. 8, 1900	27.64	29.78	2.14	936	1009	73
Aug. 14, 1901	28.72	30.16	1.44	973	1021	48
Sept. 11,1903	28.84	30.12	1.28	977	1020	43
June 17, 1906	28.91	29.98	1.07	979	1015	36
Sept.17, 1906	28.98	30.38	1.40	981	1029	48
Sept.27, 1906	28.50	30.07	1.57	965	1018	53
Oct. 18, 1906	28.84	29.80	.96	977	1009	32
July 21, 1909	28.31	30.27	1.96	959	1025	66
Sept.20, 1909	28.94	30.30	1.36	980	1026	46
Oct. 11, 1909	28.30	30.07	1.77	958	1018	60
Oct. 17, 1910	27.80	29.19	1.39	941	989	48
*Oct. 18, 1910	28.33	29.77	1.44	959	1008	49
Aug. 28, 1911	28.92	30.10	1.18	979	1019	40
Sept. 3, 1913	28.81	29.98	1.17	976	1015	39
Aug. 17, 1915	28.01	29.65	1.64	949	1004	55
Sept.29, 1915	27.70	30.14	2.44	938	1021	83
July 5, 1916	28.38	30.03	1.65	961	1017	56
Aug. 18, 1916	28.00	30.77	2.77	948	1042	94
Oct. 18, 1916	28.76	30.20	1.44	974	1023	49
Sept.28, 1917	28.48	29.88	1.40	964	1012	48
Sept. 9, 1919	27.44	29.73	2.29	929	1007	78
*Sept.14, 1919	P_o near 28.0 inches					
Sept.21, 1920	28.93	29.90	.97	980	1013	33
June 22, 1921	28.17	30.03	1.86	954	1017	63
Oct. 25, 1921	28.29	29.59	1.30	958	1002	44
Aug. 25, 1924	28.70	30.33	1.63	972	1027	55
*Aug. 26, 1924	28.70	29.62	.92	972	1003	31
Oct. 19, 1924	28.70	29.82	1.12	972	1010	38
*Oct. 20, 1924	28.83	29.62	.79	976	1003	27
Dec. 2, 1925	28.95	29.90	.95	980	1013	33
July 28, 1926	28.34	29.91	1.57	960	1013	53
Aug. 25, 1926	28.31	30.35	2.04	959	1028	69
Sept.18, 1926	27.59	29.99	2.40	934	1016	82
*Sept.20, 1926	28.20	30.13	1.93	955	1020	65
Oct. 20, 1926	27.52	29.97	2.45	932	1015	83
Sept.16, 1928	27.62	30.38	2.76	935	1029	94
June 28, 1929	28.62	29.97	1.35	969	1015	46
Sept.28, 1929	28.15	30.08	1.93	953	1019	66

Table I - Characteristics of United States Hurricanes

c	t_c	P_a		STATION	r_a
kt	hrs.	in.	mb.		n.mi.
15	4	28.28	958	Savannah, Ga.	2
7	4	28.65	970	Moss Point, Miss.	13
21	4	28.33	959	South Island, S. C.	1
11	4	29.12	986	Jacksonville, Fla.	23
18	4	28.90	979	Charleston, S. C.	50
10	4	28.48	964	Galveston, Tex.	17
14	4	29.42(1)	996	New Orleans, La.	45
7	3	29.47	998	Tampa, Fla.	14
12	4	29.46	998	Jupiter, Fla.	29
16	4	29.50	999	Columbia, S. C.	28
16	4	28.50	965	Ship at Scranton, Miss.	5
6	4	29.26	991	Jupiter, Fla.	33
12	4	29.00	982	Bay City, Tex.	16
11	4	29.23	990	New Orleans, La.	43
10	4	28.36	960	Sand Key, Fla.	7
11	4	27.80	941	S.S.Jean Nr. Tortugas, Fla.	0
11	4	28.94	980	Tampa, Fla.	45
8	4	29.02	983	Savannah, Ga.	6
16	4	29.36	994	Raleigh, N. C.	6
11	4	28.14	953	Velasco, Tex.	11
10	4	28.01	949	New Orleans, La.(Pauline St.Wharf)	12
25	3	28.38	961	Ft. Morgan, Ala.	32
11	4	28.00	948	Santa Gertrudis, Tex.	6
21	4	28.76	974	Pensacola, Fla.	0
13	3	28.51	966	Pensacola, Fla.	12
8	2	27.44	929	Mean of 2 ships and Dry Tortugas, Fla.	0
20	4	28.65	970	Corpus Christi, Tex.	-
28	3	28.99	982	Houma, La.	10
11	4	29.37	995	Houston, Tex.	33
10	4	28.29	958	Tarpon Springs, Fla.	1
22	4	28.80	975	Hatteras, N. C.	28
29	4	28.71	972	Nantucket, Mass.	12
8	4	-	-	Nr. Dry Tortugas, Fla. (2)	(b)
6	4	29.10	985	Miami, Fla.	0
14	4	29.17	988	Wilmington, N. C.	35
8	4	28.80	975	Merritt Island, Fla.	11
10	4	28.31	959	Houma, La.	3
17	4	27.61	935	Miami, Fla.	4
7	4	28.20	955	Perdido Beach, Ala.	1
16	4	29.16	988	Key West, Fla.	60
13	4	27.62	935	West Palm Beach, Fla.	3
15	2	29.12	986	Pt. O'Conner, Tex.	13
10	4	28.18	954	Long Key, Fla.	7

DATE	P_o in.	P_n in.	P_n-P_o in.	P_o mb.	P_n mb.	P_n-P_o mb.	R n. mi (a)
•Sept.30, 1929	28.80	29.96	1.16	975	1015	40	58
Aug. 13, 1932	27.83	30.11	2.28	942	1020	78	12
Aug. 4, 1933	28.80	29.96	1.16	975	1015	40	24
Aug. 23, 1933	28.63	29.48	.85	970	998	28	54
Sept. 4, 1933	27.98	29.98	2.00	948	1015	67	13
Sept. 5, 1933	28.02	30.24	2.22	949	1024	75	30
Sept.16, 1933	28.25	29.82	1.57	957	1010	53	42
June 16, 1934	28.52	29.94	1.42	966	1014	48	37
Sept. 2, 1935	26.35	29.92	3.57	892	1013	121	6
•Sept. 4, 1935	28.71	29.89	1.18	972	1012	40	51
Nov. 4, 1935	28.73	-	-	973	-	-	-
July 31, 1936	28.46	30.00	1.54	964	1016	52	19
Sept.18, 1936	28.53	29.42	.89	966	996	30	34
Sept.21, 1938	27.86	29.52	1.66	943	1000	57	50
Aug. 7, 1940	28.76	29.75	.99	974	1008	34	11
Aug. 11, 1940	28.78	30.02	1.24	975	1017	42	26
Sept.23, 1941	28.31	29.66	1.35	959	1004	45	21
Oct. 7, 1941	28.98	30.19	1.21	981	1022	41	18
Aug. 30, 1942	28.07	29.64	1.57	951	1004	53	18
July 27, 1943	28.78	30.02	1.24	975	1017	42	17
Sept.14, 1944	27.88	30.66	2.78	944	1038	94	49
•Sept.14, 1944	28.31	29.39	1.08	959	995	36	26
Oct. 18, 1944	28.02	29.80	1.78	949	1009	60	27
•Oct. 19, 1944	28.42	29.67	1.25	962	1004	42	34
Aug. 27, 1945	28.57	30.13	1.56	968	1020	52	18
Sept.15, 1945	28.09	30.00	1.91	951	1016	65	12
Sept.17, 1947	27.76	29.83	2.08	940	1010	70	19
•Sept.19, 1947	28.53	29.70	1.17	966	1006	40	28
Oct. 15, 1947	28.59	29.65	1.06	968	1004	36	13
Sept.21, 1948	27.62	29.61	1.99	935	1003	68	7
•Sept.22, 1948	28.41	29.83	1.42	962	1010	48	16
Oct. 5, 1948	28.85	29.77	.92	977	1008	31	27
Aug. 24, 1949	28.86	30.20	1.34	977	1023	46	24
Aug. 26, 1949	28.16	30.12	1.96	954	1020	66	22
Oct. 3, 1949	28.45	29.95	1.50	963	1014	51	15
Aug. 30, 1950	28.92	29.71	.79	979	1006	27	21
Aug. 30, 1954	28.35	-	-	960	-	-	-
•Aug. 31, 1954	28.38	-	-	961	-	-	45
Sept.11, 1954	27.97	29.77	1.80	947	1008	61	37
Oct. 15, 1954	27.66	29.55	1.89	937	1001	64	18
Aug. 12, 1955	28.40	29.77	1.37	962	1008	46	45
Sept.19, 1955	28.51	29.87	1.36	966	1012	46	50
Sept.23, 1956	28.76	29.83	1.07	974	1010	36	29

Table I - Continued

t_c hrs.	P_a in.	P_a mb.	STATION	r_a n.mi.
4	28.80	975	Panama City, Fla.	6
4	27.83	942	East Columbia, Tex.	0
4	28.98	981	Brownsville, Tex.	15
4	28.66	971	Cape Henry, Va.	16
4	27.98	948	Jupiter, Fla.	0
4	28.07	951	Brownsville, Tex.	2
4	28.25	957	Hatteras, N. C.	7
4	28.52	966	Jeanerette, La.	0
4	26.35	892	Long Key, Fla.	0
-	29.18	988	Columbia, S. C.	30
-	28.73	973	Miami, Fla.	0
4	28.46	964	Ft.Walton, Fla.	0
4	28.52	966	Mean of 2 ships off Hatteras, N.C.	0
4	28.04	950	Hartford, Conn.	7
4	28.87	978	Port Arthur, Tex.	5
4	28.78	975	Savannah WBO, Ga.	2
3	28.66	971	Houston WBO, Tex.	0
4	29.00	982	Carrabelle, Fla.	0
4	28.10	952	Seadrift, Tex.	4
4	28.78	975	Ellington Fld., Tex.	2
4	27.97	947	Hatteras, N.C.	14
4	28.31	956	Pt. Judith, R. I.	3
4	28.02	949	Dry Tortugas, Fla.	4
4	28.42	962	Sarasota, Fla.	1
4	28.57	968	Palacios, Tex.	1
4	28.09	951	Homestead, Fla. (FECRR)	0
3	27.97	947	Hillsboro, Fla.	8
4	28.57	968	New Orleans, WBO, La.	2
4	28.76	974	Savannah, Ga., WBAS	7
4	28.45	963	Key West WBO, Fla.	8
4	28.47	964	Clewiston, Fla.	8
4	28.92	979	Miami, Fla.	2
4	28.86	977	Diamond Shoals L/S, N. C.	3
3	28.17	954	W. Palm Beach WBAS, Fla.	0
4	28.88	978	5 mi. west Freeport, Tex.	7
4	28.92	979	Ft. Morgan, Ala.	0
4	28.35	960	Aircraft Recon.	
4	28.42	962	Suffolk County AFB, RI.	
6	27.97	947	40°N - 71°W Aircraft Recon.	0
4	27.70	938	Little River, N. C.	4
4	28.40	962	Ft. Macon, N. C.	0
4	28.63	970	Cherry Point, N. C.	18
4	28.78	975	Destin, Fla.	2

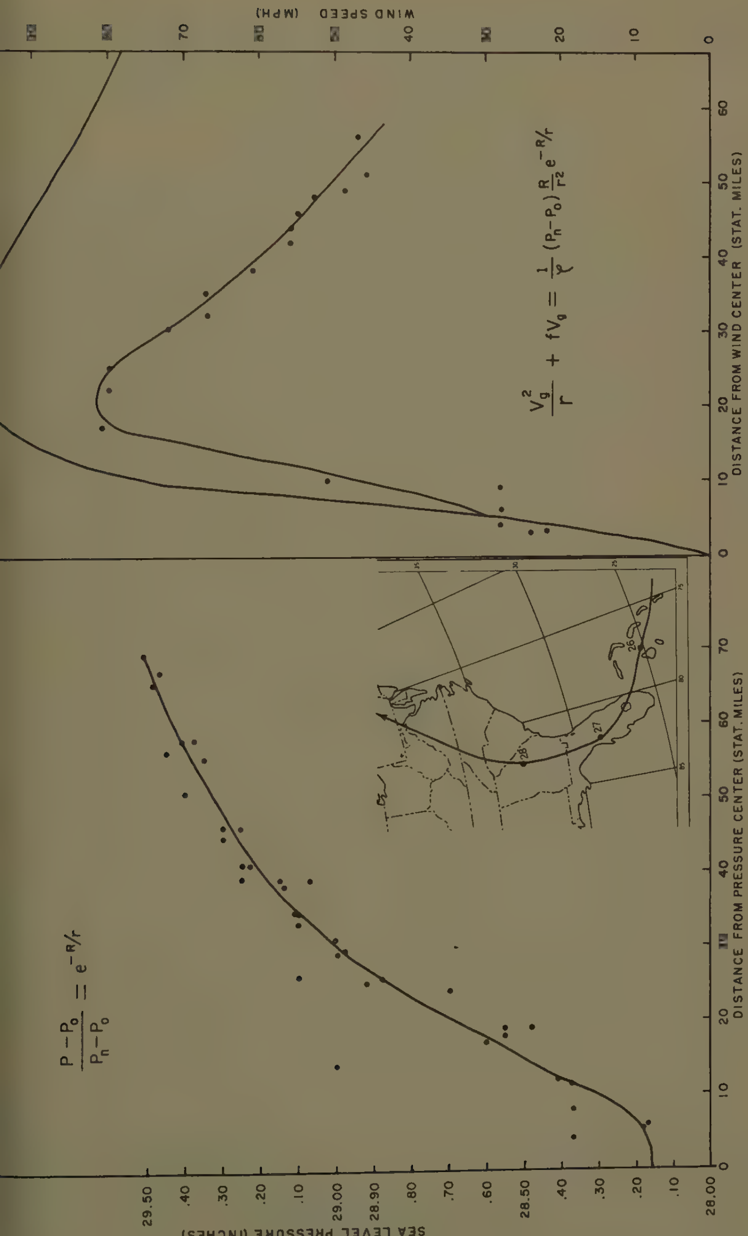
Footnotes for Table I

Note: All values are estimated except p_a.

p_o	central pressure
p_n	asymptotic pressure
$p_n - p_o$	pressure difference between "outside" of storm and center
R	radius to region of maximum wind speed (a) computed from pressure profile (b) observed from wind speed record
c	forward speed of the storm
t_c	time interval over which storm movement is averaged to obtain c
p_a	lowest pressure detected by a barometer

Station - at which p_a was observed

r_a	minimum distance from station to track of storm center
*	same hurricane as previous line, entering or passing coast at a second point
(1)	Lowest in region where pressure profile parameter were compute[d] 29.31 inches observed at Mobile, Ala. later
(2)	Parameters obtained by interpolating is time between ship off western end of Cuba and Miami, Fla. and apply to vicinity of Dry Tortugas.



Hurricane Aug. 26-27, 1949

Fig. 1. Pressure and Wind distribution in model hurricane.

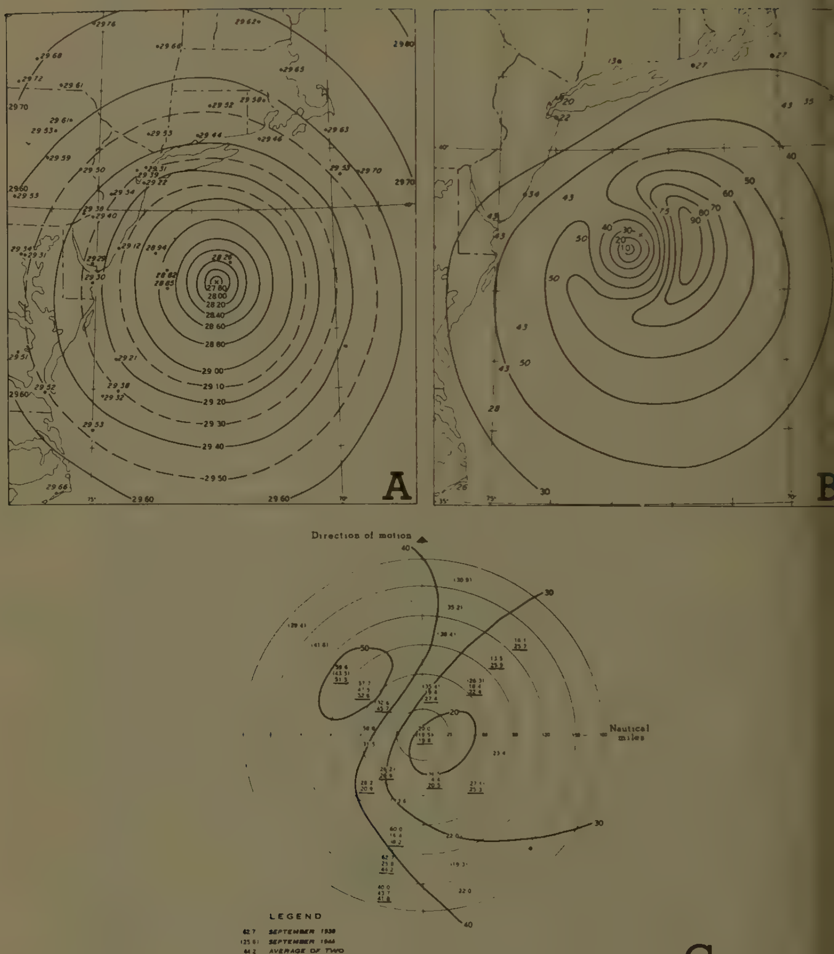


Fig. 2. Reconstructed surface pressure, wind speed and deflection angl
for 1200 EST in the hurricane of September 21, 1938,
After Myers and Jordan.
(A) Surface Pressure, (inches of Mercury)
(B) Wind Speed (M.P.H.). X is location of pressure center
(C) Deflection Angle (degrees)

(4) it is assumed that the winds will blow parallel to the isobars. A more histicated theory, as well as observations show that the wind must cross isobars toward lower pressure. The angle of inflow, that is the angle with ch the wind crosses the isobars, must be asymmetric with respect to the ter of a moving storm. The best determination of the angle of inflow for v England hurricanes is shown as Fig. 2c.(8)

Tides and Storm Surges

f the effects of wind waves and swell are eliminated, the resulting ele-on of the sea surface is usually controlled by the gravitational attraction he sun and moon and the topography of the region of observation. Minor nges of a seasonal nature are also produced by the seasonal changes in ospheric pressure, prevailing winds, and temperature of the water. The cts of these gravitational and climatic factors can be computed many nths in advance and in this paper are identified as the normal tide.

n the neighborhood of an intense storm, the actual level of the sea will er significantly from the normal tide, and the actual level of the sea may referred to as a storm tide. The difference between the storm tide and the mal tide is called the storm surge.

Economically important storm surges may be produced by extratropical rms as well as hurricanes. Extratropical storms have been responsible the highest storm tides north of Cape Cod and at a few more southerly lo-ons. However, the surge produced by many hurricanes is much higher n those produced by any extratropical storm. Fig. 3 shows the maximum orded tide at a representative number of places along the Atlantic and Gulf sts of the United States.(9) The remainder of this paper will be devoted ropical cyclones and the storm tides produced by them.

The peak water level reached in a hurricane will always be somewhat ater than the storm tide level because of the effect of waves with periods nly a few seconds. It is desirable to separate these two phenomena be-se their effects are quite different. If the peak tide level is two feet above n sea level, it will penetrate inland to the contour which is approximately feet above mean sea level, even if this distance is several miles, and it not rise appreciably above two feet regardless of the slope of the beach. A two foot wave, on the other hand, may run up to a sloping beach to a ht of seven or eight feet above the still water line, but under most con-ons the effects of such a wave cannot penetrate inland more than a wave th from the still water line, and most of the damage from such waves will confined to a much narrower trip. The waves which accompany the land-of a hurricane will generally be in the range of 100 to 1000 feet in length. remainder of this paper will be restricted to a discussion of the storm .

n most cases the principal cause of the storm surge is the frictional drag veen the wind and water. Changes in atmospheric pressure, although usu-of secondary importance, may occasionally be more important than the l effect.(10) Waves breaking against the shore or over barrier reefs may contribute to the piling up of water near the shore.

Fig. 4 shows the storm surge produced by Hurricane CAROL at 19 Coast Geodetic Survey tide stations. The dots on the curves for New London, port and Woods Hole indicate approximate surge heights after the tide

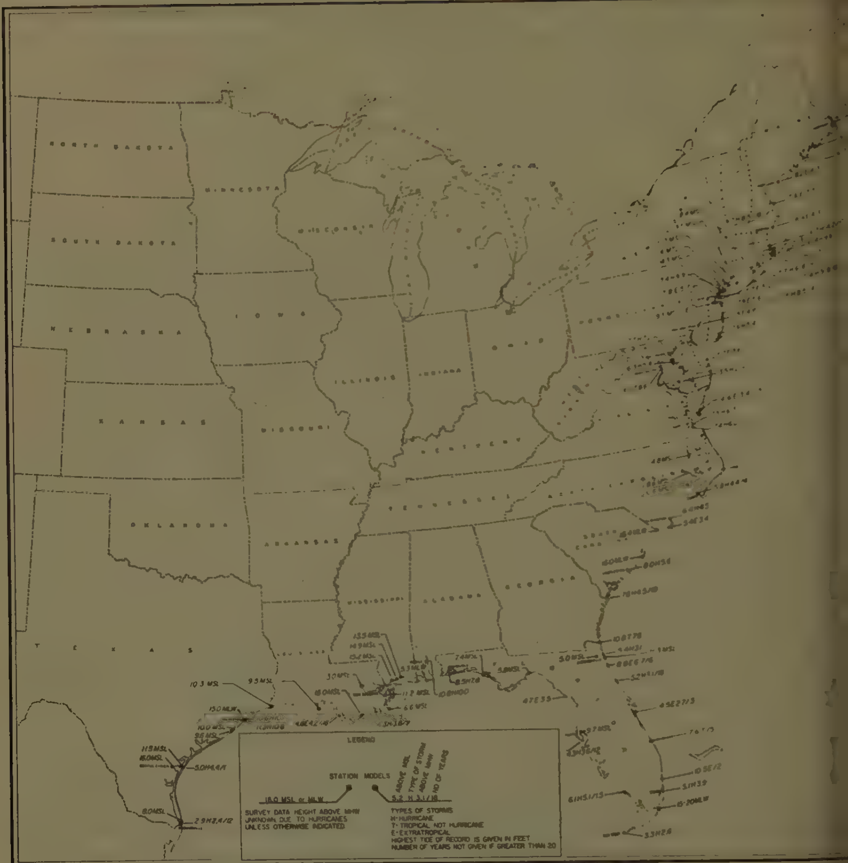
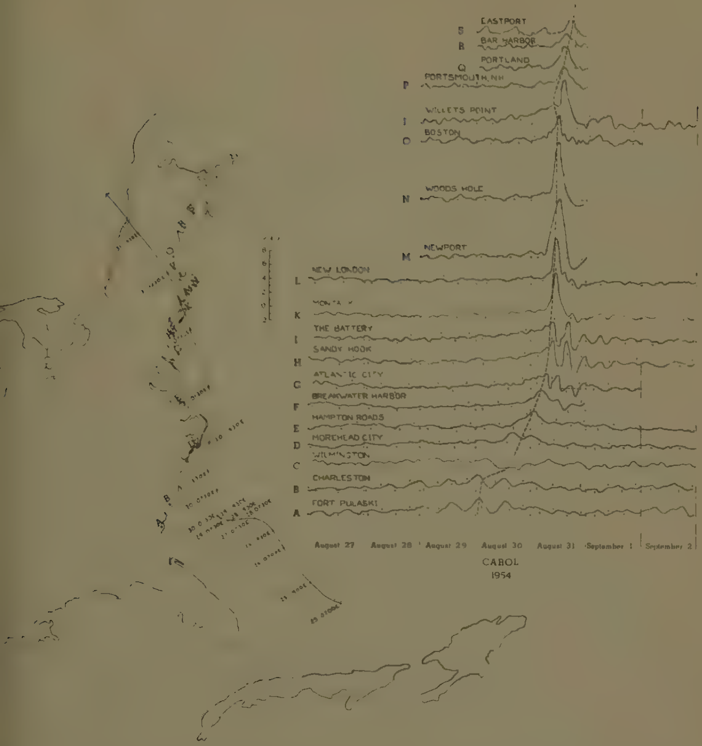


Fig. 3. High tide data for stations having a record for 20-years or longe

gages became inoperative due to the storm. The dashed line crossing the storm surge curves indicates the time at which the storm was nearest eac tide station. The storm track is shown to the left of the storm surge curv The letters on the map and beside the curves identify the storm surge cur with the geographical location of the tide gage. The maximum surge heigh and mean tide range are shown on the left.

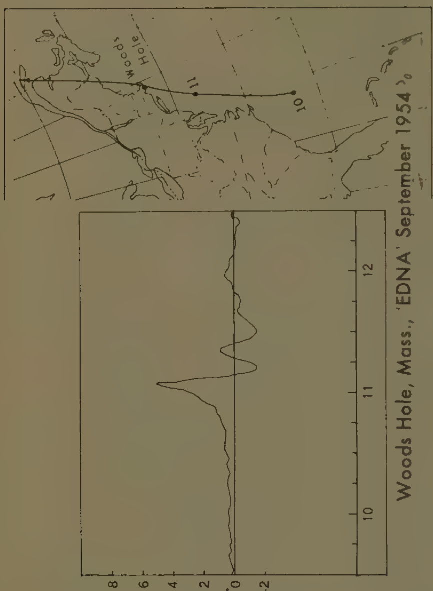
With the exception of Willets Point, near the western end of Long Island Sound, the maximum surge height at all stations between Hampton Roads a Cape Cod occurred between the passage of the storm and two hours later. Willets Point, the maximum surge occurred more than four hours after th passage of the storm center. Thus, the water continued to rise at many pl to the left of the storm track after the wind had acquired an offshore component, suggesting that the surge arrives as a wave which had been genera elsewhere. This is particularly true at Willets Point. The notion of a wa receives further support from the damped oscillations readily seen in the records for Atlantic City, Sandy Hook, and the Battery, and suggested in th



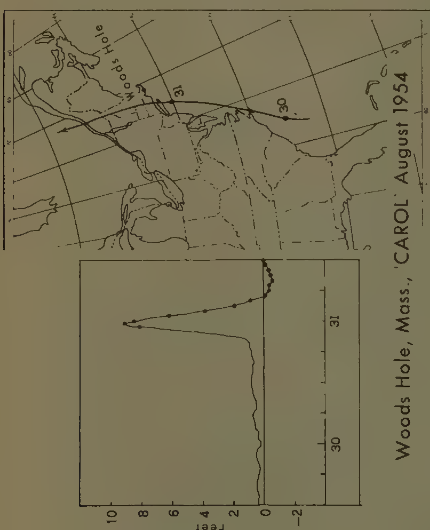
Max. Surge	Mean Tide Range
1.9	18.2
2.1	10.4
3.3	8.9
2.8	7.8
7.4	7.2
3.8	9.5
9.1	1.8
8.5	3.5
7.3	2.6
6.5	1.9
3.2	4.4
3.5	4.6
2.6	4.1
2.9	4.2
2.7	2.5
2.3	2.8
1.0	3.7
2.3	5.1
2.2	6.9

ig. 4. Track of hurricane and storm surge curves for hurricane CAROL, 1954.

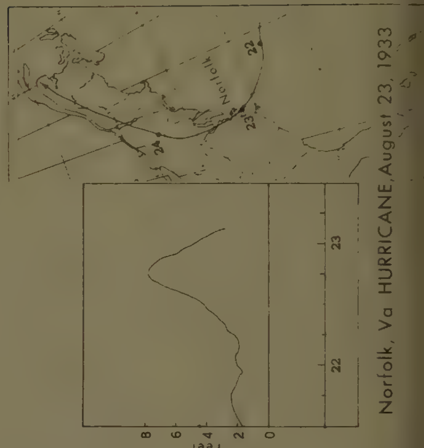
ords of several other stations. These oscillations, called resurgences by field,(4) are characteristic of the hurricane surge in this region.
Fig. 5 shows the storm surge as a function of time at the recording tide ion nearest the point of maximum storm surge height for several hurri-es. A trend toward rising or falling sea level may become evident 12 to hours before the arrival of the storm. In either, a rapid rise begins when storm is about 100 to 200 miles distant, that is when the wind speed reach-about 30 miles per hour with the approach of the storm. This period of d rise lasts from two to six hours and is generally followed by a slightly e rapid drop to something below the normal water level in areas with good nage. In marshland or other areas with poor drainage, the water may re-n well above normal for several days. Fig. 6 shows the contours of the m surge along the coast for four hurricanes. Notice the tendency for the value to be higher on the right or to extend farther from the center on right hand side of the storm track. Notice also that the peak storm tide be as much as 10 feet higher than the storm tide 60 miles to either side e peak.



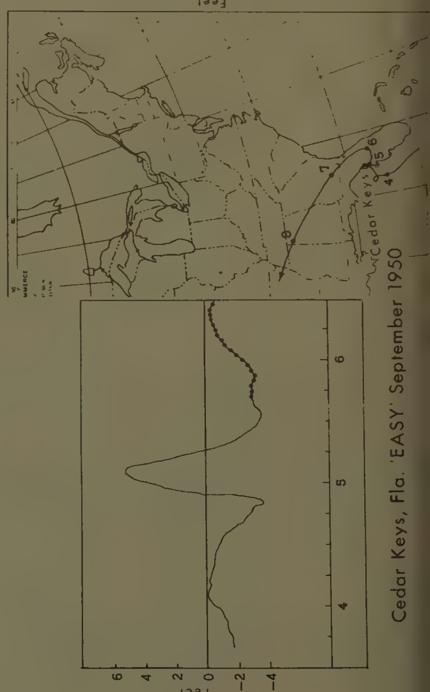
Woods Hole, Mass., 'EDNA' September 1954



Woods Hole, Mass., 'CAROL' August 1954



Norfolk, Va HURRICANE, August 23, 1933



Cedar Keys, Fla. 'EASY' September 1950

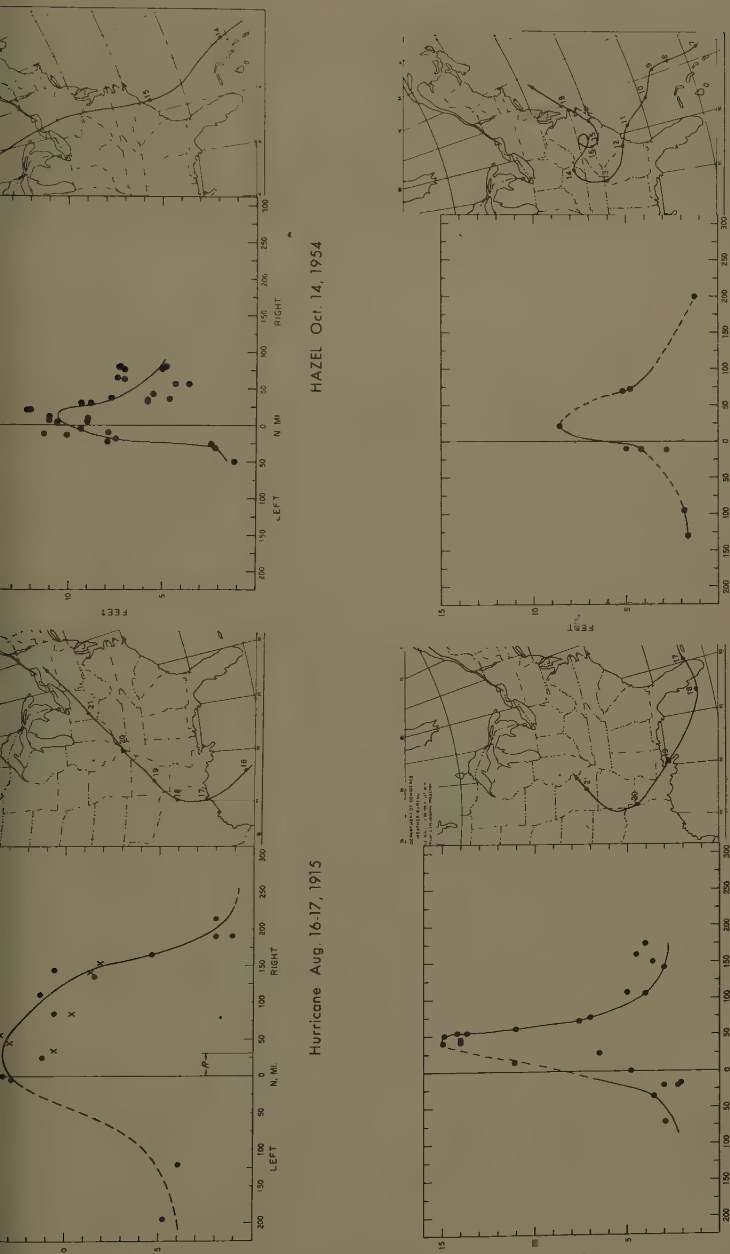


Fig. 6. Storm surge profiles along the coast for four hurricanes.

Fig. 7 based on data furnished by the Corps of Engineers, Coast and Geodetic Survey, Woods Hole Oceanographic Institution and others, shows the storm surge heights produced by Hurricane CAROL at a great many locatio along the southern New England and Long Island coasts. Here the effects of topography and local wind conditions are shown by the higher surge heights near the heads of converging bays and at exposed points, and the decreased surge heights near the widest section of Long Island Sound. The varied effe of topography and local wind conditions make it difficult to construct graphs such as Fig. 6, or to develop any relation between storm surge height and meteorological parameters. It is clear that one should avoid using surge heights obtained near the heads of estuaries in developing such relationship but it is not possible to avoid all local effects.

Maximum Storm Tide as a Function of Storm Intensity

The effect of wind in piling up water is nearly proportional to the wind stress. The wind stress is generally taken as being proportional to the squa of the wind speed, and according to Eqs. (2) and (4), this is approximately p portional to the pressure deficiency in a hurricane. The direct hydrostatic fect of the decreased pressure is also proportional to the decrease. Thus, are encouraged to look for a correlation between $(P_n - P_o)$, or P_o alone, an the maximum storm surge, that is the maximum effect of the storm on the level of the sea. The results of such an effort are shown in Fig. 8.

The highest observed tides along the Gulf Coast of the United States duri 30 hurricanes is indicated by dots as a function of the central pressure of th storm as it crossed the coast. An attempt has not been made to eliminate t normal tide, as the normal tide range is less than two feet in most of this region, and sufficient data for the elimination of the normal tide were not available for all of the reports. The regression equation based on these dat gives a correlation coefficient of 0.68. The extreme differences between th storm and the normal tide for 10 Atlantic Coast hurricanes are indicated by x's. The tide range along the Atlantic Coast is much greater than in the Gu of Mexico, and a correction for the normal tide is necessary in order to obtain any useful degree of correlation. Largely because the variability due t the normal tide is eliminated, the correlation coefficient for Atlantic data h been increased to 0.72 even though these data were not considered in the de vation. Consideration of this single parameter is thus sufficient to account for half of the observed variability in maximum storm surge heights.[12] N improvement was obtained by considering P_n and R as defined in Eqs. (3) a (4), or by using the storm tide potential as defined by Reid.[6]

Effects of the Forward Motion of the Storm

Proudman[13] using a very elementary theory, has shown that the respon of the water in a rectangular canal to a moving atmospheric disturbance co be expressed in the form:

$$h = (1 - V_s^2/gD)^{-1}\ \bar{h} \qquad ($$

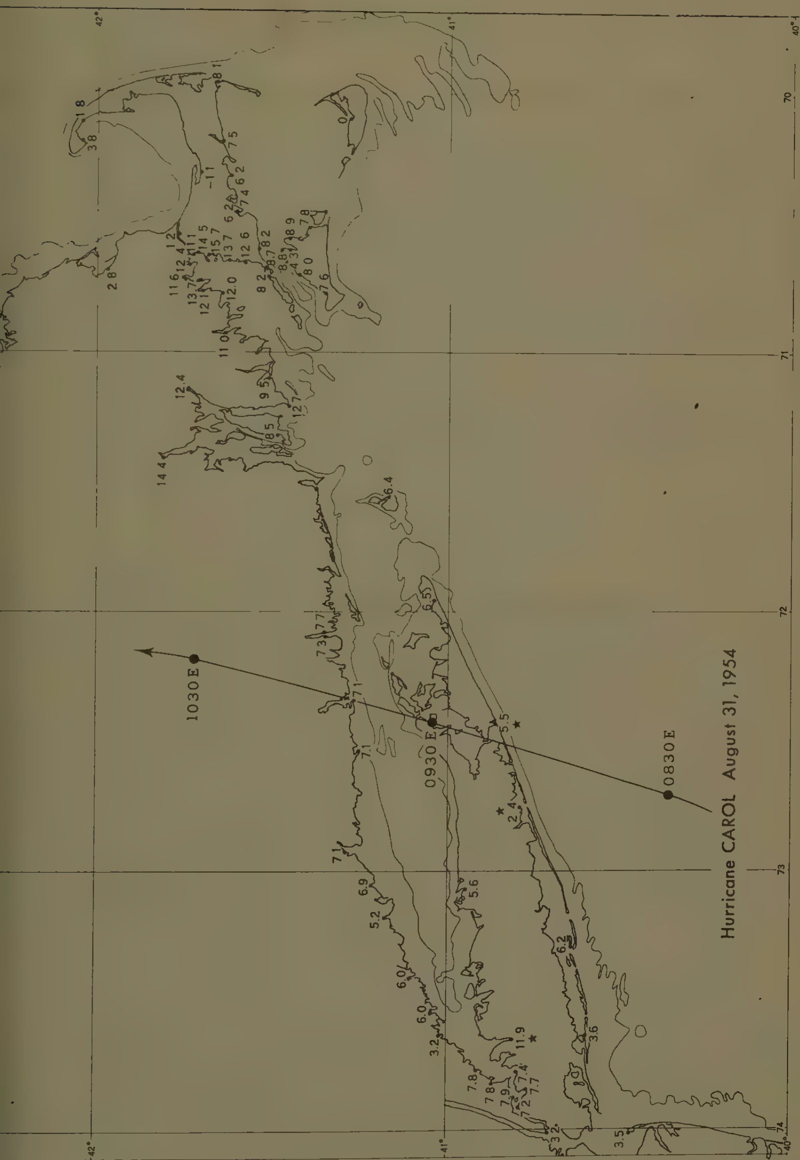
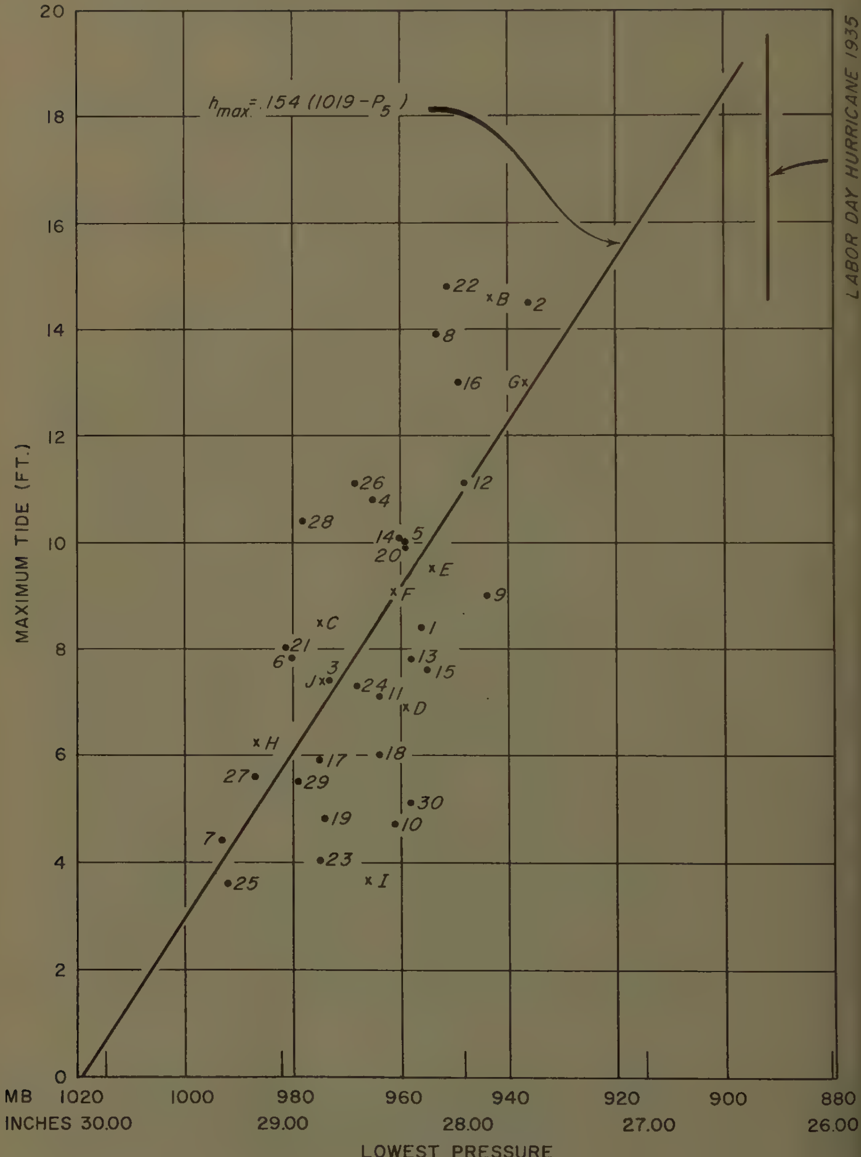


Fig. 7. The storm surge associated with the landfall of Hurricane CAROL, 1954.


$h_{max} = .154\,(1019 - P_5)$
LABOR DAY HURRICANE 1935
MAXIMUM TIDE (FT.)
20
18
16
14
12
10
8
6
4
2
0
MB 1020 1000 980 960 940 920 900 880
INCHES 30.00 29.00 28.00 27.00 26.00
LOWEST PRESSURE
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
B
C
D
E
F
G
H
I
J

h = height of water level disturbance

$\bar{h}$ = Atmospheric pressure disturbance expressed as the height of a water barometer

V_s = speed of atmospheric disturbance

D = depth of canal

g = acceleration of gravity

ter writers(14) have shown that these results are modified somewhat, when e effects of the earth's rotation and the two dimensional nature of the dis-rbance are considered. The essential feature of the development which rries over to more complex situations is, that for any basin there is a criti-l velocity of motion of the atmospheric disturbance which will produce maxi-um response in the water. An examination of the residuals obtained from g. 8 supports this concept, but there are not sufficient data from any type of astline to establish the proper form of this relationship from empirical idence.

The speed of the storm can become important in quite another way, for the eed of the storm, together with the radius of maximum winds determines e duration time and the length of the fetch over which the storm winds can t to produce storm tides.

Hurricane Frequency

From a practical point of view, it is necessary to consider the frequency of orms as a function of intensity, size, and region of occurrence. Any real ends in such frequencies would also be highly significant in an economic nse. Unfortunately, the amount of data available for making such studies is ther limited and all data are not of the same quality. The nature and sense the available data can be indicated by three figures adapted from recent eather Bureau reports to Congress.(16)

Fig. 9 shows the frequencies of tropical storms and hurricanes for the riod 1887-1956. A tendency for years of above normal hurricane frequency be followed by a period of below normal hurricane frequency can be easily ted. There is no evidence of any well marked trend or regular cycle in rricane frequency. It may be possible to obtain a useful indication of the mber of storms to be expected during the next two or three years by con-lering the number which have occurred during the last 10 to 20 years, but best estimate of hurricane probabilities more than a very few years in the ure would be obtained from considering the frequencies for the entire peri- of reliable records.

Fig. 10 shows the number of times destruction has been caused by tropical orms on the United States mainland during the present century.

Fig. 11 shows the accumulated frequencies of tropical storms, whose ntral pressures were 29.00 inches of mercury or less, for several sections the U. S. Coast. Cape Hatteras is taken as the dividing line between the rth Atlantic and South Atlantic sections of the coast for this figure. The a for the Florida Keys are clearly different from those of the other regions. ither the size of the sample available, nor the present state of theoretical owledge concerning hurricanes is sufficient to determine whether the

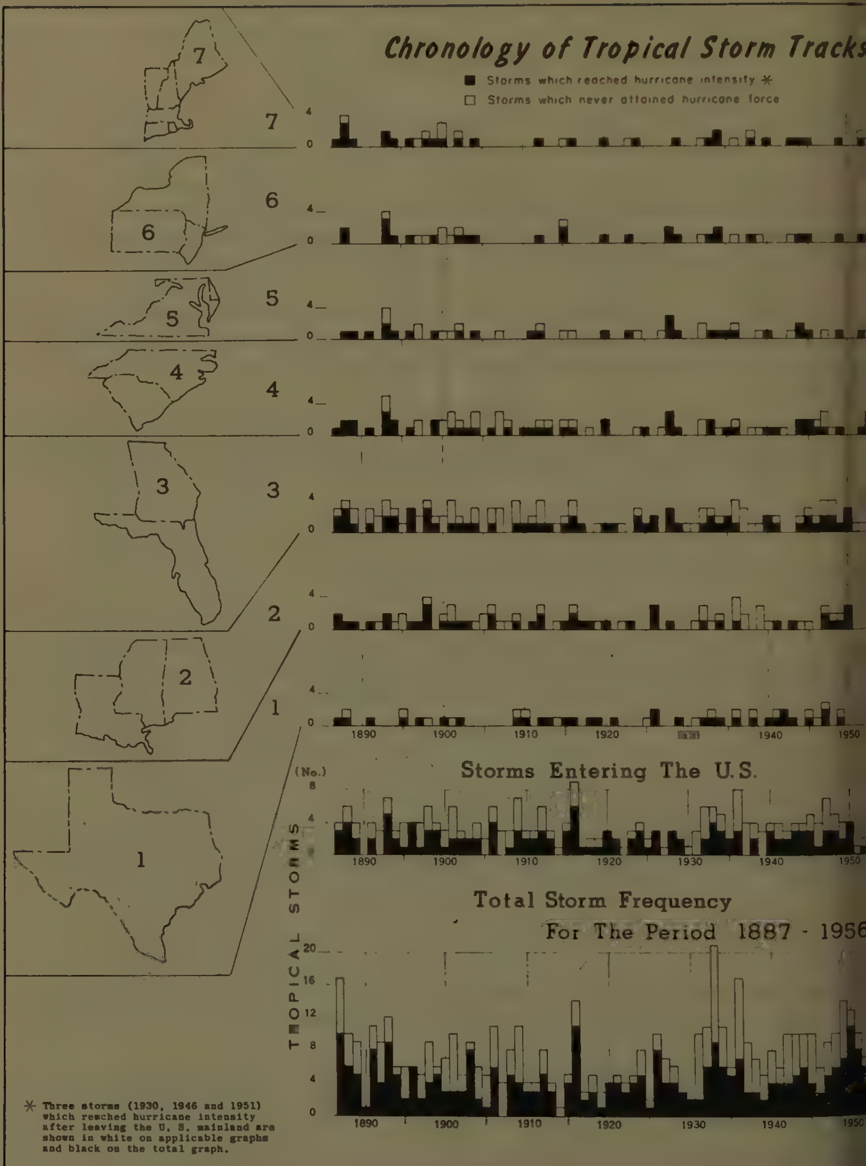


Fig. 9. Chronology of storm tracks. The occurrence of hurricanes and tropical storms in the different coastal regions shown at left, and the total occurrence.

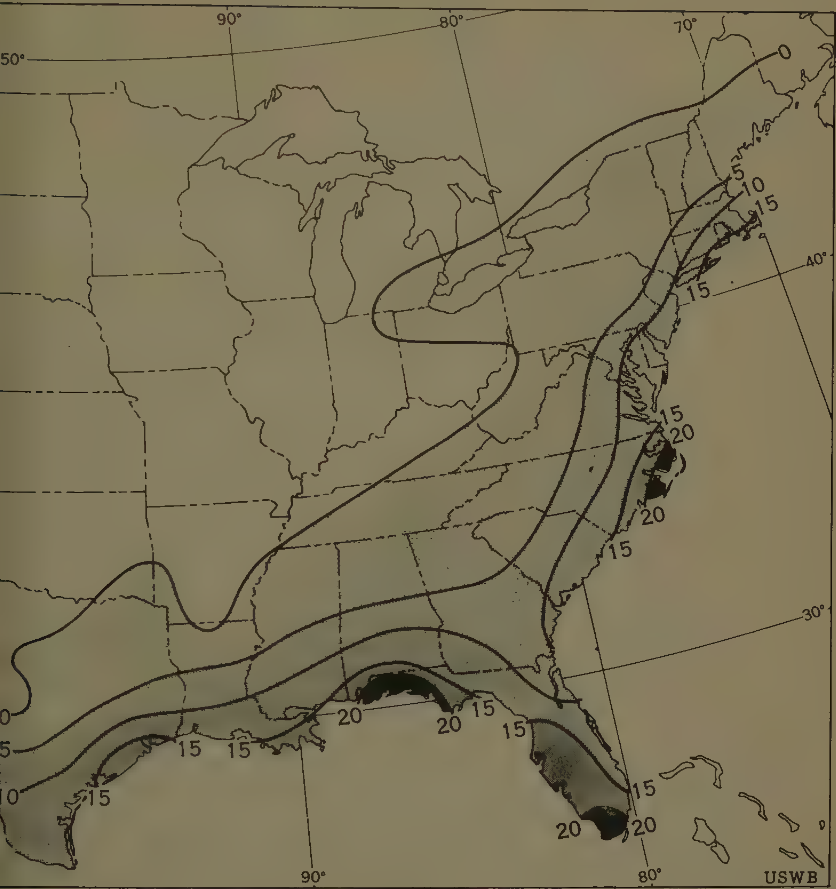


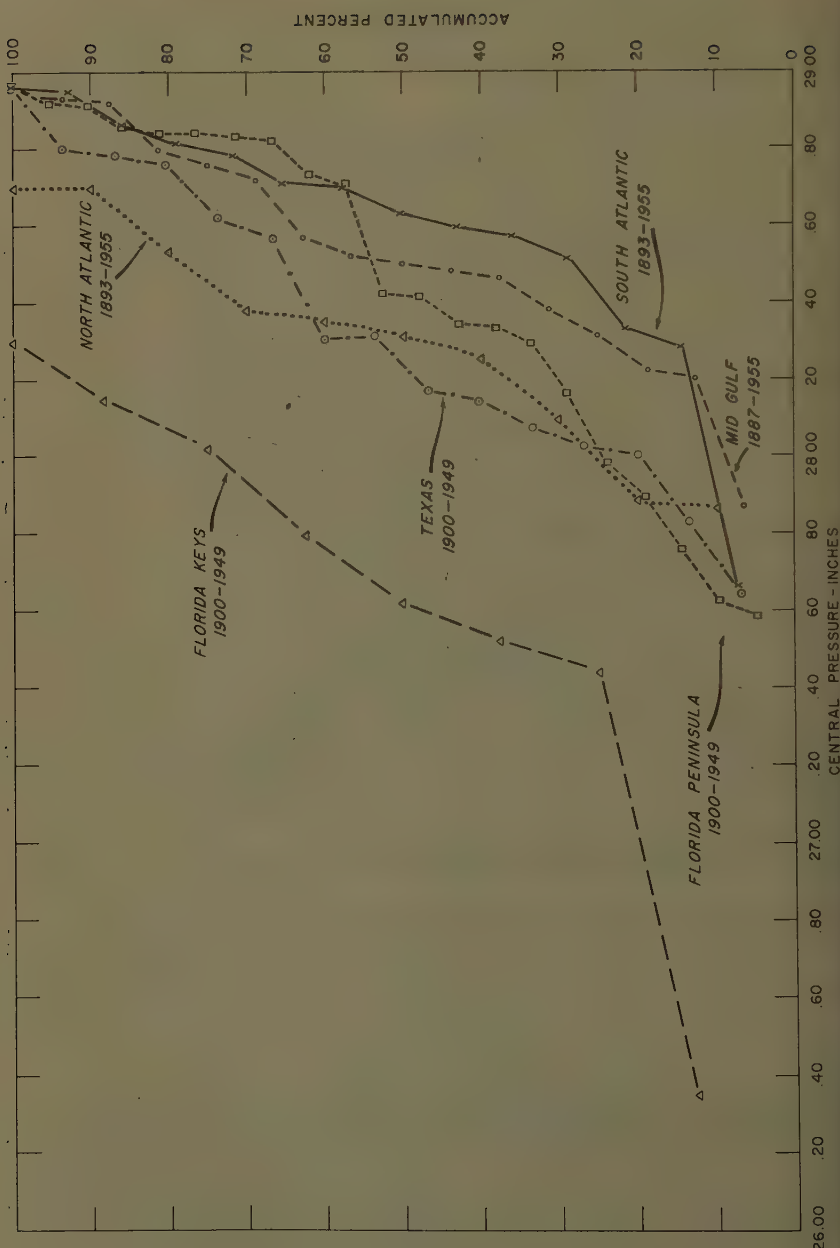
Fig. 10. Number of times destruction was caused by tropical storms, 1901-1955.

fferences in the other five curves are due to sampling error or to some ndamental aspect of hurricane behavior.

Hurricane Observations and Forecasts

If a hurricane approaches land in the neighborhood of a powerful weather dar installation, such as that now operating at Cape Hatteras, N. C. and on der for about 15 other coastal sites, an experienced radar hurricane ob- rver can locate the center of the storm within 5 to 10 miles while the storm nter is within 150 miles of the station. The accuracy decreases as the stance from the radar station is increased.

When the storm is out of range of land based radar, the position of the orm center must be determined from the available ship and aircraft reports.


ACCUMULATED PERCENT
CENTRAL PRESSURE - INCHES
NORTH ATLANTIC 1893-1955
SOUTH ATLANTIC 1893-1955
MID GULF 1887-1955
TEXAS 1900-1949
FLORIDA KEYS 1900-1949
FLORIDA PENINSULA 1900-1949
100 90 80 70 60 50 40 30 20 10 0
29.00 .80 .60 .40 .20 28.00 .80 .60 .40 .20 27.00 .80 .60 .40 .20 26.00

casionally the eye of the storm will pass directly over a ship. Somewhat ore frequently, but not on every day of the hurricane's existence, an airplane l penetrate the eye of the hurricane. In these cases the storm center can en be located within an accuracy of about 10 miles. In the more usual case, storm center must be located in the center of an isobaric system de- mined from fewer than a dozen ship reports, or from radar observations de from aircraft on the edge of the storm. The error in locating the storm nter in this way will frequently be as much as 20 or 30 miles and occasion- y as much as 60 miles. Errors in communication via radio and teletype- iter compound the uncertainties resulting from the currently unavoidable ortcomings in our observational procedures.

A few months after the storm, when mailed in reports, free of communi- tion errors, as well as much additional information not available to the fore- ster during the storm are obtained, it is possible to repeat the analysis and termine the position of the storm track with an accuracy impossible during storm.

A simple extrapolation of the storm path accounts for considerably more n half of the obtainable skill in hurricane forecasting. Even the more phisticated forecast methods, based on a knowledge of flow patterns at all els, give only a velocity and acceleration of the storm movement, and per- uates any existing error in the storm position.

When all of these factors are considered, it is evident that storm vements cannot be forecast with timetable accuracy. A study of forecast curacy for the period 1952 to 1955 indicated an average error of 69 nautical les in the 12-hour forecast of storm position.[15] This was made up of an erage error of 83 miles during the period 1952-1953 and 63 miles in 1954- 55. A gradual improvement in forecast accuracy is to be expected as the sult of improved instrument and forecast techniques.

A hurricane watch is announced for specific areas whenever it is recognized t a specific hurricane threat exists for any region of the coast. The hurri- ne watch is announced before it is possible to delineate the area in which storm is likely to go inland and when the hurricane effects are believed to at least 24 hours away from the coast. A hurricane warning is issued for pecific area whenever there exists a reasonable probability that the storm l go inland in that region within the next 24 hours. When the average accu- cy of forecasts and the length of coastline seriously affected by a hurricane considered, it is evident that the hurricane warnings must cover an area veral times as large as that which will actually suffer severe damage from storm.

SUMMARY

A full realization of the hurricane's potential for danger and of the difficulty predicting its future behavior exactly, should materially reduce the loss of and property due to these storms.

More than 390[1] persons lost their lives in Hurricane AUDREY, largely cause they had lived through other hurricanes with comparatively little mage and did not believe that this storm could be as severe as predicted.

Hurricanes vary in intensity. Tropical storms which barely qualify as rricanes may elevate the sea level no more than four feet, but severe hurri- es may elevate the sea level by 15 feet or more above normal tide for the

time of the hurricane landfall. Intense extratropical storms may produce rises of five to six feet above normal.

The sea level may be raised by one or two feet for a thousand miles of coastline by even a moderately intense hurricane, but the really severe sto surges of more than 5 feet above the normal tide level rarely extend over more than 100 miles of coastline.

The duration of the rapid rise in sea level is rarely greater than 6 hour and in areas with good drainage the fall is even more rapid. In areas of po drainage, moderately high water may persist for days after the storm.

Knowledge of the intensity of a hurricane before it reaches the coast is meager. The average error in a 12-hour forecast is approximately 60 naut cal miles, and if warnings are to be effective, an area several times as lar as that actually affected must be alerted.

Disasters such as that which occurred with Hurricane AUDREY can be eliminated by the evacuation of the coastal plain in the threatened region to elevation of 10 to 20 feet above the normal high water level. In marshland, where high ground may be as much as 40 to 100 miles away by highway, ma mass evacuations may be required to avoid one catastrophe. With improve instrumentation and forecasting techniques, the number of evacuations, late proved to be unnecessary, will be reduced, but no technique now available w permit the prediction of hurricane motion with timetable accuracy.

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Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

CONTENTS

DISCUSSION
(Proc. Paper 1880)

MECHANICS OF SEDIMENT-RIPPLE FORMATION[a]

Corrections by J. Bogardi

. BOGARDI.[1]—Corrections to Discussion by J. Bogardi:

'age 1832-3, 14th line "... difficulties to Mr. Liu—of the multitude ..." ıld be "difficulties to Mr. Liu to determine these correlations—of the titude."
'age 1832-4, Eq. (8) "$T_c = U*^2 =$ " should be "$T_c = \rho U* =$ ".

'orrections to Closure by H. K. Liu:

'age 1832-29, 16th line "... This factor is proportional to gd/U_*^2..." ıld be "The third parameter β is proportional to $d/DS = gd/U_*^2$, the ınel stability factor, which is the inverse ...".
'age 1832-29, delete (a) the words "in the form" before Eq.(35), (b) Eq. , and (c) the line following Eq. (35), and substitute the following:
If one uses the channel stability factor $d/DS = gd/U_*^2$ the configuration of oed is quantitatively defined by the parameter β mentioned above unique- The relationship of β and gd/U_*^2 according to Bogardi is

$$\beta = \frac{gd}{U_*^2 d^N} \tag{35}$$

'rom the data of Fig. A, Bogardi assumes that $N \cong 0,882$.
ogardi further states: Obviously there is a parameter α proportional to $'\nu$, which defines the bed configuration uniquely too and

$$\alpha = \frac{d}{\left(\frac{U_* d}{\nu}\right)^{\frac{2}{3-N}}} \tag{36}$$

'inally Bogardi pointed out that a third parameter ϵ proportional to U_* rally defines the bed configuration uniquely too:

$$\varepsilon = \frac{U_*}{d^{\frac{1-N}{2}}} \tag{37}$$

, α and ϵ are obviously interrelated, and their value depends upon the erature, as well as on the specific gravity of the sediment. Assuming a erature of 20° C., and a specific gravity of 2,65, the parameter values obtainable directly from Eqs. (35), (36), and (37).

ig. 19 shows after Bogardi the $U_* = \epsilon d^{\frac{1-n}{2}}$ equations.

'roc. Paper 1197, April, 1957, by H. K. Liu.
sst. Prof. Technical Univ. of Budapest, Hungary.

According to Fig. 19 ripples are only possible if $1{,}777 < \epsilon < 4{,}06$ and $d < 2mm$. Bogardi constructed similar figures for Eqs. (35) and (36)."

NORTHEASTERN FLOODS OF 1955: RAINFALL AND RUNOFF[a]

Discussion by Howard M. Turner

HOWARD M. TURNER,[1] M. ASCE.—This is a very interesting engineering cription of the floods of August and October, 1955. The data selected and presentation of it give an excellent summary of the magnitude and extent his storm, which, as the author points out, exceeded previous records of nsity of rainfall and flood flows and covered a tremendously wide area. writer has made some further comparisons of the magnitude of this flood 1 other large floods in New England.

Fig. 1 shows area-depth curves for total rainfall in various New England rms showing how the 1955 storm exceeded previous records even of the

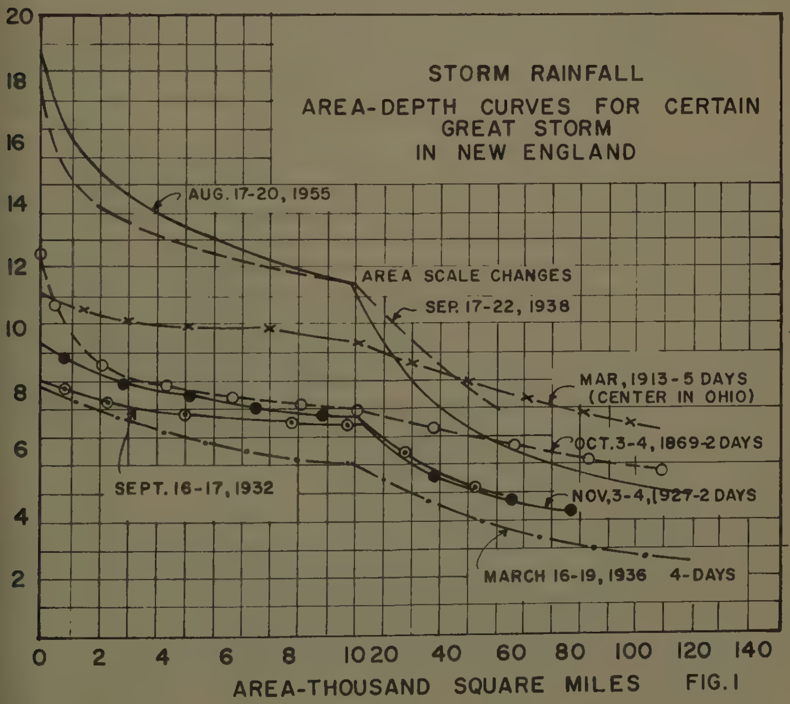


FIG. 1

Proc. Paper 1662, June, 1958, by Tate Dalrymple.

Cons. Engr., Boston, Mass.

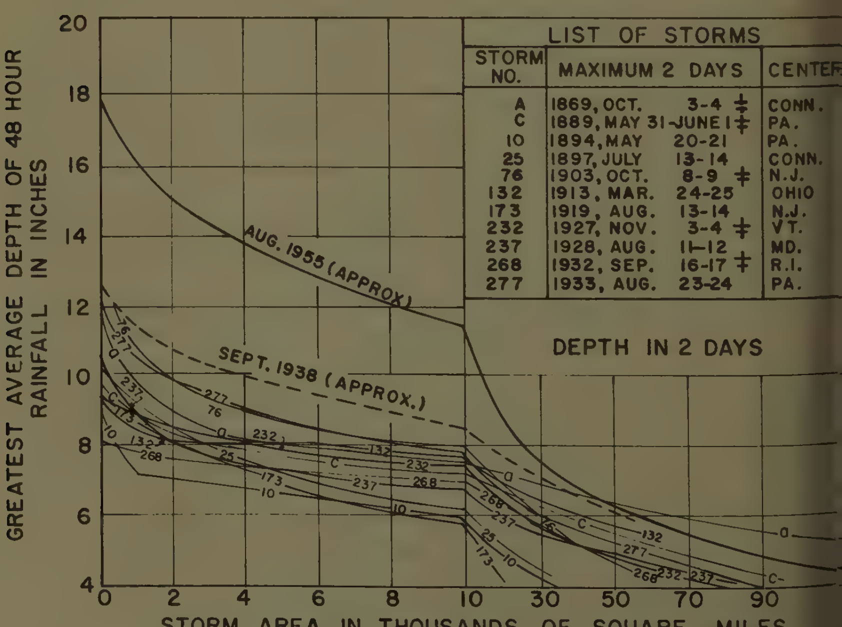
very large four-day hurricane storm from September 17-21, 1938. Most of the 1955 storm came the two days of August 18 and 19.

The Storm Rainfall report of the Miami Conservancy District, Revised, 1936,(1) gives area-depth curves of great storms in the northeastern part of the country from 1869 to 1933. On Fig. 2 the 1938 and 1955 hurricane storm are plotted on the two-day area-depth curves for the Northeast taken from that report showing how much the 1955 storm exceeded any of these record storms.

It is interesting to compare past analyses of storm intensities with the later records and with the 1955 storm and to note how the results have changed by records which have been added through the years. As an exampl Fig. 3 shows a curve of high rainfall frequency taken from "The Frequency of High Rates of Rainfall" by Allen Hazen, published in 1921.(2) The writer has plotted on this David L. Yarnell's values, published in 1935(3) for 10-year and 100-year maxima estimated for Massachusetts and also the maximum actual rainfalls for different periods of time recorded during the 1955 storm. The results of the latter would require an entire recasting of these frequencies derived thirty-six years ago.

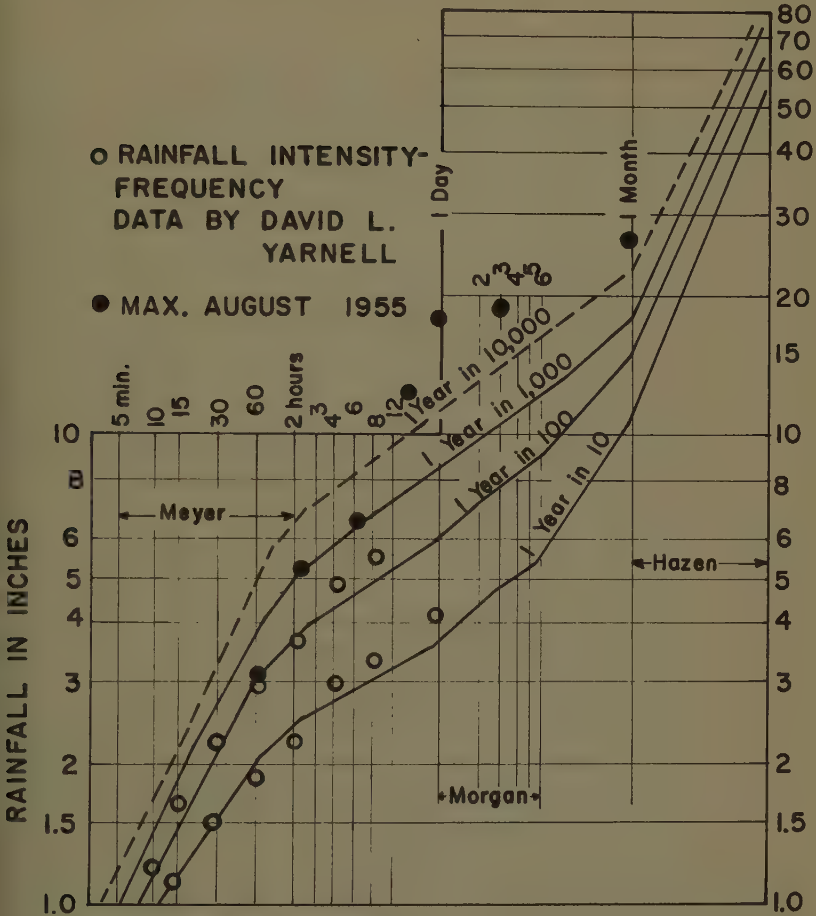
In a paper of "Flood Flow Formulas based on Drainage Basin Characteristics" by H. B. Kinnison and B. R. Colby,(4) a frequency curve of storm rai fall was included. This is shown on Fig. 4. On this are plotted the maximur records for the 1955 storm showing the extreme magnitude of this storm co pared with that curve.

It is naturally expected that a longer period of records would have the re sult of increasing the maximum results obtained, but it is interesting to note



TIME-AREA-DEPTH CURVES FOR 2 DAY RAINFALL

FIG. 2

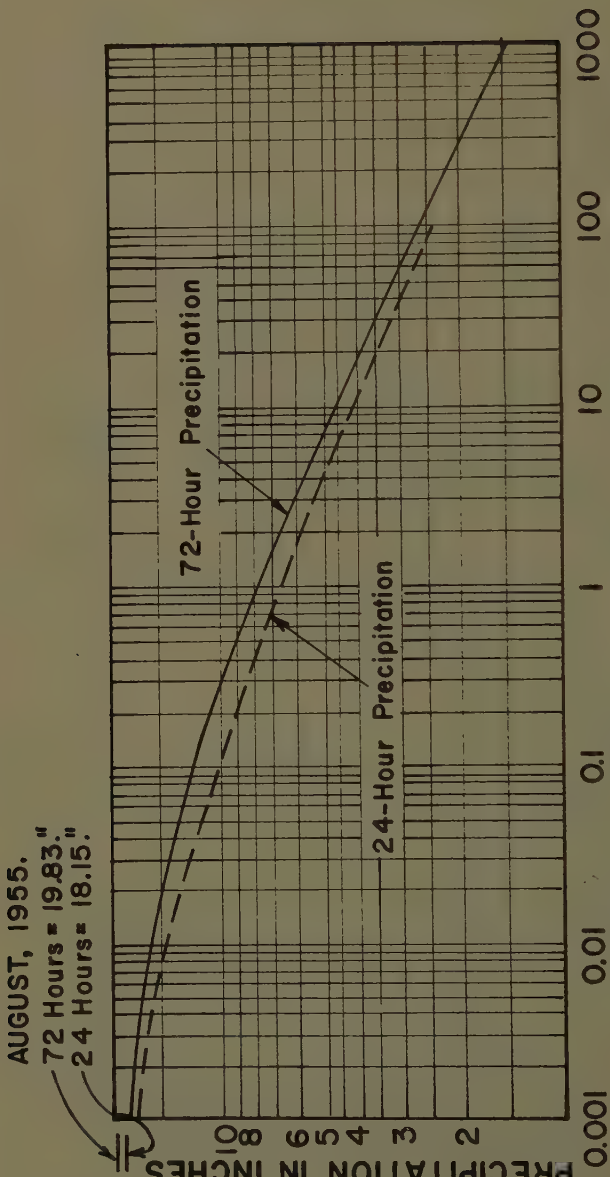


CURVES OF RAINFALL INTENSITY DATA FROM "THE FREQUENCY OF HIGH RATES OF RAINFALL" BY ALLEN HAZEN—1921

FIG. 3

the present period does not seem to show any orderly increase as a long-ime of records alone would cause. Fig. 5 shows records of large three-storm rainfall since 1855. The two largest storms have come in the last ears, exceeding by 25 per cent anything of record before.

Considering flood flows, records of the gage heights on the lower necticut River at Hartford are available for the last 114 years. There are a few high-gage records going back for 318 years. The greatest flood in 1936; the next, in 1938; and now the third, in 1955. As far as can be rmined from storm and flood histories, nothing of these recent large nitudes occurred in over 300 years. Fig. 6 shows this large flood record e Connecticut River at Hartford back to 1683. From all that can be



PERCENTAGE PROBABILITY OF OCCURRENCE, IN A YEAR.

FREQUENCY OF 24 & 72 HOUR PRECIPITATION FOR DRAINAGE BASINS IN MASSACHUSETTS.

CURVES FROM "FLOOD FORMULAS BASED ON DRAINAGE BASIN CHARACTERISTICS": H.B. KINNISON AND B.R. COLBY.

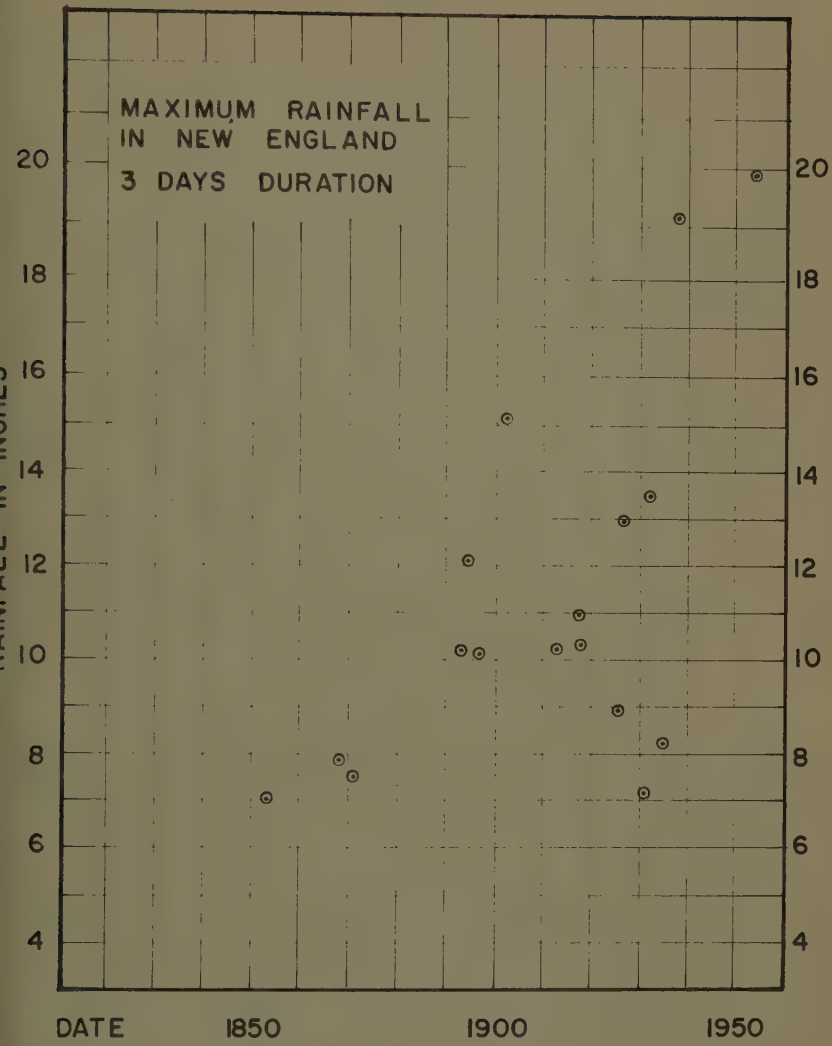


FIG. 5

ermined, the three largest floods have occurred in the last twenty years. great size of the 1955 storm is shown by the large flood it caused on this er though it was largely confined to the lower third of the drainage area.

The author has shown the relation between the peak flow and drainage as on his Table 5 and Fig. 6. The writer has made up Fig. 7 showing the same data for New England, including the large floods previous to that of ust, 1955. A curve of the 40% Myers rating is shown as an enveloping ve including practically all records up to that time. The 1955 flood re-es a 70% rating curve to include the high records of the 1955 flood for the nage areas of 300 to 700 square miles.

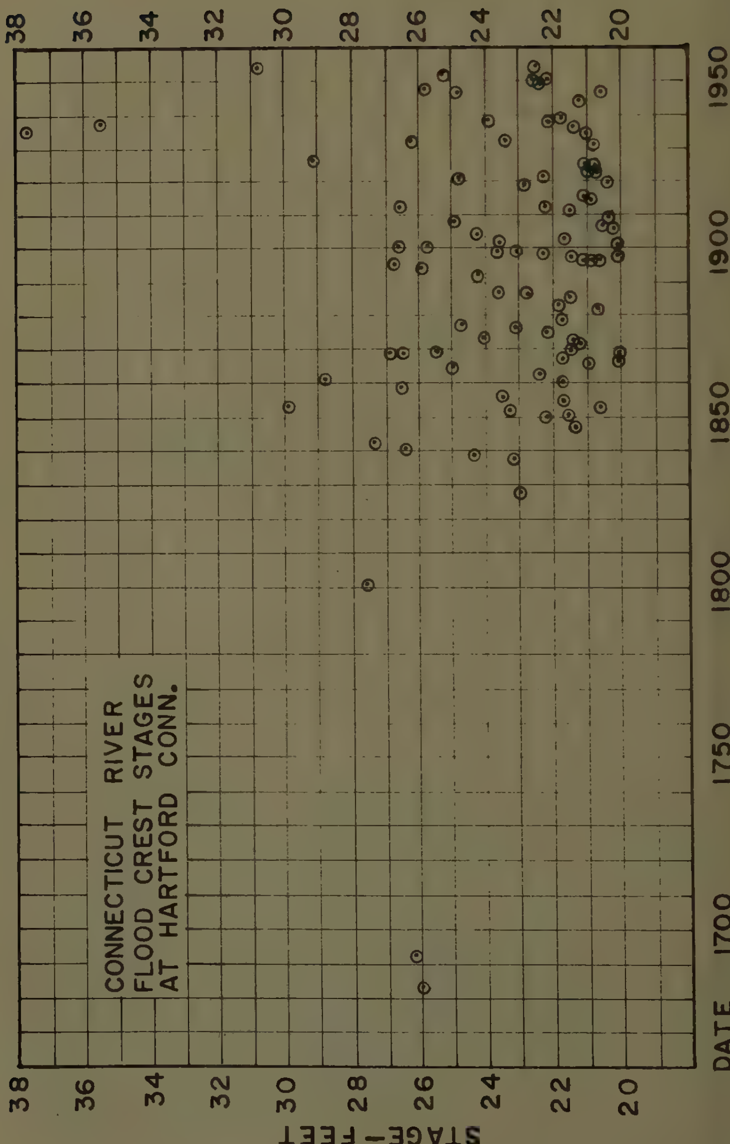
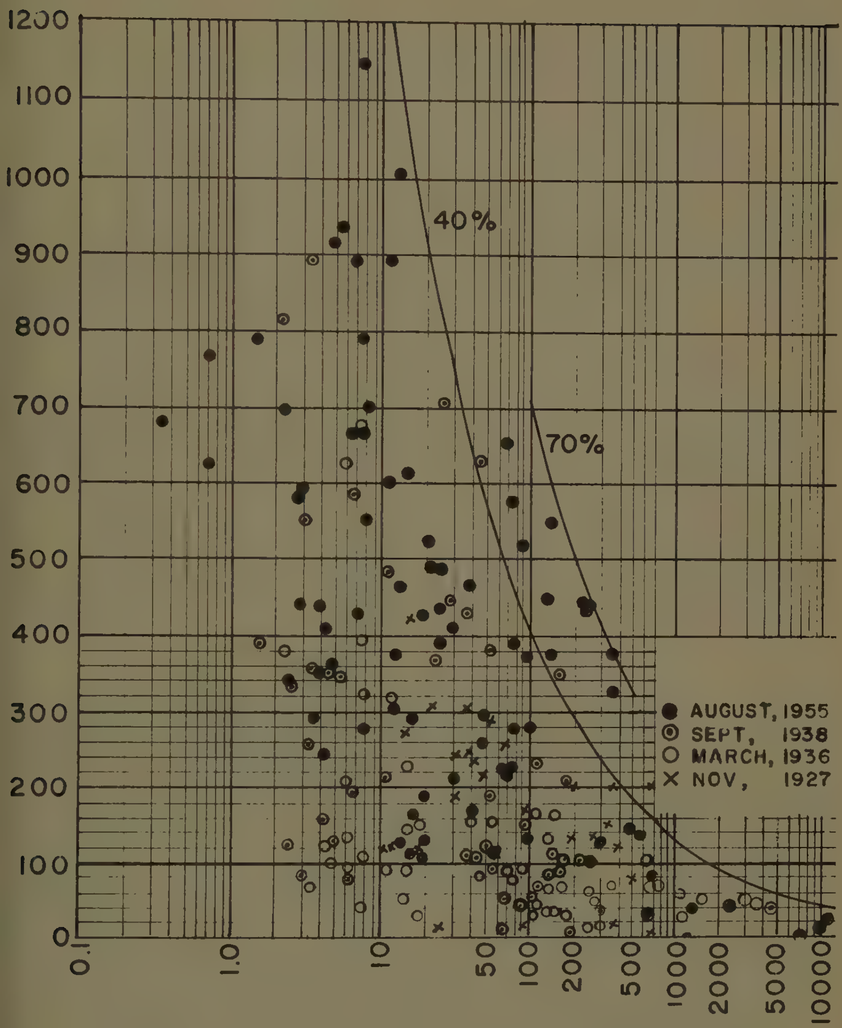

CONNECTICUT RIVER
FLOOD CREST STAGES
AT HARTFORD CONN.
STAGE—FEET
38 36 34 32 30 28 26 24 22 20
DATE
1700 1750 1800 1850 1900 1950

FIG. 6



DRAINAGE AREA IN SQUARE MILLS
PEAK FLOWS OF NEW ENGLAND STREAMS
FIG. 7

These very high rates of flow on these fairly small drainage areas shown in the Myers rating and in the rainfall frequency studies are due to the heavy rainfall in a short time. The author shows on Fig. 3 the rainfall curve at Hartford with the total of 14.4 inches in a total time of about 42 hours. Actually 97 per cent of it fell in 28 hours; and about 83 per cent of it, in 24 hours

Runoff

The author gives some figures of the percentage of runoff to rainfall, which, except for a few cases, appear to be somewhat lower than might be ex pected from a flood of this intensity. Some of the very high ones that are given may very likely have been affected by the failure of dams.

The author has noted the rapid rise and fall of the hydrographs of this flood. In the case of the large rivers the reason for this is quite evident. For example, on the Connecticut River the heavy storm flows were confined to the lower third of the drainage area. Channel storage was thus confined t only the lower portion of the river valley, and the recession curve of the hydrograph representing the drainage of the channel storage would be shorte in duration as the amount of storage to drain away would not be so great. Th same thing shows in the recession limb of the hydrograph of the Delaware River at Port Jervis, where the heavy storm rainfall was concentrated into about 50 per cent of the drainage area.

The hydrographs of the smaller rivers remain to be analyzed. The write believes, however, that the 6- or 12-hour unit hydrographs for these rivers will prove to be close to the corresponding unit hydrographs for the 1938 flood except that the peaks may be somewhat higher.

Frequency

The question of frequency is one of the most difficult problems in flood studies, but some frequency rating of the size of a given flood seems to be r quired. The author gives a tabulation and a frequency curve for the Northeast, according to the United States Geological Survey's method of rating the recurrence interval of any given flood as its ratio to the mean annual flood, but the curve gives the recurrence interval up to 100 years only, not long enough for many requirements. This curve shows very different results fro that developed for Connecticut by B. L. Bigwood and M. P. Thomas.(5)

The section on flood frequency of the 1942 report of the Committee on Floods of the Boston Society of Civil Engineers, published in 1942,(6) analyz the frequency of floods on the Connecticut River at Thompsonville, where there were then over 93 years of record. Various methods were used. The results, using data up to 1935, i.e., prior to the 1936 and 1938 floods, gave a frequency of the 1936 flood from 10,000 to 16,7000 years, depending on the method used. The same computations made with three years' more data including these two large floods gave frequencies from 300 to 400 years.

Apparently the same difficulty is still found by the United States Corps of Engineers as shown in its recent report on the 1955 flood,(7) where for four rivers in southern New England the flow for a given frequency used in 1956 was much higher than shown by the curves tentatively adopted prior to the 1955 flood. See Table 1.

The paper by Kinnison and Colby, published in 1945,(4) which has been us a great deal in Massachusetts, rates floods as "minor," "major," and "rare with stated frequencies of 15, 100, and 1,000 years respectively. On the riv

TABLE I - 1955 FLOOD
COMPARATIVE FLOOD PEAKS AND FREQUENCIES

River	Drainage Area sq. mi.	Maximum Discharge 1938 c.f.s.	Maximum Discharge 1955 c.f.s.	Ratio Peak Discharge to mean annual flood 1955	Kinnison-Colby[4] Major 100-year c.f.s.	Kinnison-Colby[4] Rare 1000-year c.f.s.	U.S. Corps of Engineers 1955 Flood[7] Frequency in Years Before 1950	U.S. Corps of Engineers 1955 Flood[7] Frequency in Years After 1955
MASSACHUSETTS AND RHODE ISLAND								
Quinebaug River at Westville	93.8	- -	17,500	22.6	5,410	8,970	- -	- -
W. Br. Farmington R. at New Boston	75.3	18,500	34,300	14.5	12,400	21,900	- -	- -
Quaboag River at West Brimfield	149	8,470	12,800	11.1	9,410	15,400	- -	- -
Blackstone River at North Bridge	139	- -	16,900	10.9	7,530	12,700	- -	- -
Kettle Brook at Worcester	31.3	1,300	3,970	7.9	2,860	5,000	- -	- -
Chicopee River at Indian Orchard	688	45,200	40,500	6.5	36,400*	60,500	- -	- -
Blackstone River at Woonsocket, R.I.	417	15,100	29,600	5.6	16,600	29,100	3,700	167
Westfield River near Westfield	497	55,500	85,000	- -	69,700	111,000	500	77

* At Bircham Bend 702 sq. mi.

River	Drainage Area sq. mi.	Maximum Discharge 1938 c.f.s.	Maximum Discharge 1955 c.f.s.	Ratio Peak Discharge to mean annual flood 1955	Kinnison-Colby Major 100-year c.f.s.	Kinnison-Colby Rare 1000-year c.f.s.	Frequency Before 1950	Frequency After 1955
CONNECTICUT								
Scantic River at Broad Brook	98.4	11,200	13,300	12.1	- -	- -	- -	- -
Quinebaug River at Putnam	331	20,900	48,000	12.0	- -	- -	See note**	250
Naugatuck River at Naugatuck	246	17,000	106,000	11.8	- -	- -	See note***	264
Willimantic River at South Coventry	123	14,900	46,700	11.5	- -	- -	- -	- -

** Curve gives 2000-year frequency for 32,000 c.f.s.
*** Curve gives 2000-year frequency for 75,000 c.f.s.

on which there are records concerning the 1955 flood, the Kinnison-Colby figures would be as shown on Table 1. These generally rate the higher 195 flood flows as equal to or exceeding a "rare" flood.

In that paper the flood frequency ratings were based upon precipitation a depth of runoff. The use of precipitation in determining flood frequency has its difficulties because of the varying percentage that occurs in the form of runoff in the flood, but the rainfall data are more inclusive. The writer be-lieves that the flood runoff figures would have some advantage as data for flood frequency computations instead of the peak floods. The runoff data ar often not so readily available and require more work to use, but the results are on a broader basis, eliminating the varying characteristics of each stream. With a runoff frequency established, the peak flood of each locatio could be determined by unit hydrograph methods. On a few of the streams for which Kinnson-Colby figures are available for the high 1955 flood areas the runoff compares to their "rare" or 1,000-year flood.

All this points out the difficulty of the problem. Each of these successiv floods gives more information with which to work; but as the author points out in his paper, figures that still show such wide variation are not very "satisfactory." Yet, it is a problem which is important. In over 950 years of records of the flow of the river Danube, the greatest flood was in 1501; t next in 1787 was 85 per cent of it; and the third one in 1899, 75 per cent of it.(1) There is no question but what these extraordinary events occur at wi intervals, and we have to know something of the degree, time, and magnitud in flood works.

Fortunately, frequency is not required so much in the design for spillwa capacities of dams except indirectly; but for other designs, such as flood-control reservoirs, flood channel improvements, and other flood-control work, some method is needed to fit the design flood to the value of the pro-posed construction. The 1955 flood has certainly given perhaps an unscien tific but in some ways very convenient yardstick. It has been taken as the design flood on some recent river-improvement designs. As far as the pub is concerned, a large storm like that of 1955 is, of course, a very good floo measure. It is much easier to describe flood-control measures for a flood intensity that has already been experienced than it is to talk about the some what mythical frequency of a 100- or 1,000-year flood, which may come in next year.

It is interesting to look back and see the tremendous development of the science of flood analysis and prediction since the 1927 flood. There are st problems, but the progress made justifies the belief that the future will sol them.

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EXPERIMENTS ON SELF-AERATED FLOW IN OPEN CHANNELS

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(Proc. Paper 1890)

SYNOPSIS

leasurements of distribution of air concentration in self-aerated flows presented in this paper. The experiments were made in a rough channel ind-grain type surface at various slopes and discharges, and the data used as a basis for study of the mechanism of entrainment of air and to te the air content and distribution to the characteristics of the flow. The ysis of the data shows that the air distribution can be adequately de-bed by relationships based upon a simplified concept of turbulent transport thus are functions of the flow characteristics. The maximum depth and nean velocity are both shown to increase above those of a corresponding erated flow.

INTRODUCTION

characteristic of high-velocity, open-channel flow is the phenomenon of aeration in which atmospheric air is insufflated into and mixed with the to create the appearance of "white water" with its violently agitated and efined free surface. This condition is frequently observed in flows down chutes and spillways.

has been reasonably well established by several observers(1,2,3) that lows over a spillway from a quiescent reservoir, incipient aeration does ccur on the slope until a point or region is reached at which the boundary thickness is equal to the depth. The characteristic roughening of the

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water surface immediately upstream of the appearance of "white water" can be readily observed in such occurrences. With increasing initial water depths, the points where the surface roughening and the "white water" occur move downstream along with the intersection of the boundary layer with the water surface. These considerations suggest that the aeration phenomena a related to the conditions of turbulence in the flow. Examination of such flow by means of high-speed photography indicates that the upper boundary of the flow is rather ill-defined. It consists of a zone which appears white because of entrained air, while above this zone a spray of water droplets occurs mov ing more or less parallel to the flow and below which is a region of discrete air bubbles suspended in the fluid. Other investigators have described air-entrained flow as being made up of air bubbles entrained in water in the low levels and water particles in the air in the upper regions. This concept is further emphasized by measurements of the air concentration distribution(4,5,6) which show that the air concentration increases continuously from the bed, exhibiting a smooth transition from a finite value near the bed to a maximum value of 100 per cent at the free surface which value is approached asymptotically.

This paper presents the results of systematic experiments on distributio of air in self-aerated flows in a channel set at various slopes and operated a various discharges. A provisional analysis is made of the data with respect to air distribution in a vertical section transverse to the direction of flow with a view of arriving at empirical relationships between concentration parameters and the flow characteristics.

Experiments and Results

Apparatus

The experiments were undertaken in a channel 50 ft. long, 1.5 ft wide, an 1 ft deep, which could be adjusted to any slope from horizontal to nearly ver cal. The width was chosen so that for ordinary flows sidewall effects due to growth of the sidewall boundary layer and air entrainment along the sides could be avoided in the central portion of the cross section. That the width was adequate to make the flow two-dimensional in the center portion was demonstrated by earlier measurements of air concentration and velocity in transverse section at the end of the flume.(4) The flow was obtained by gra ty from the laboratory main supply flume through two feed lines and control by hydraulically operated valves. The water reached the inlet of the flume through hollow support members and through a rectangular conduit on the underside of the channel. Fig. 1 shows the channel and appurtenances.

At the inlet the flow is guided through two 90-degree vaned bends and a contraction. The inlet gate itself is cantilevered from its mounting at the to of the flume so that there are no slots or guides to disturb the flow. Furthe guidance is provided by a lip attached to the bottom of the gate and extendin inward to further contract the flow; therefore, the gate opening corresponds to the depth of flow at the upstream end of the channel. The maximum gate opening is 0.5 ft. Pitot measurements of the flow just downstream of the in with an 0.25-ft gate opening showed no deviation greater than 2 per cent fro the mean velocity except in the bottom corners where slightly smaller veloc ties were encountered.

In the experiments here described, the air-entrainment process was intensified by the installation of artificial roughness on the channel bed that w

siderably rougher than the painted steel sidewalls of the channel itself. material used for the roughness was a commercial non-slip fabric coat-with granular particles. The particles, whose mean size was 0.028 in., a mean spacing of 0.039 in. and were embedded in a mastic, as shown in . 2.

The discharge rate was measured by two calibrated Pitot cylinders mount-n the supply lines and leading to differential gages mounted on the control el. Each discharge meter was calibrated separately for flows from 1/2 0 cfs with an accuracy of the order of 1 per cent.

A control panel contained the actuating components for the hydraulic tem and indicators for the measurement of slope, gate opening, and dis-rge which permitted adjustment of these variables to predetermined ies.

The electrical air-concentration measuring instrument has been fully de-ibed in an earlier report.(7) The method consists basically of a measure-it of the difference between the conductivity of a mixture of air and water the conductivity of the ambient water alone. A strut supporting a pair of bes is combined with the electrical circuit so that the air-concentration isurements may be made in a small region of the flow. The probes were in. wide and 1/4 in. apart. The instrument is direct reading, and the ition between conductivity and air concentration can be determined ana-cally. The strut holding the probes is arranged so that it is possible to verse the flow cross section both vertically and laterally. The instrument imited to measurements not closer than 0.02 ft from the walls because of rent deflections by the walls at lesser distances. Since the measurements he air concentration depend upon the resistance or specific sensitivity of suspending water, the meter was zeroed in water continually circulated n the test channel through a deaerating tank, thus providing water of the ie character as that in the flume. Oscilloscope measurements show a idly fluctuating current between the probes. Since the meter could not ow these rapid fluctuations, the value read on the meter represented an rage value. This average compared well with results obtained by means mechanical sampler which had been previously developed at the Labora- and which was not subject to the same averaging problems. A photograph he probes and strut supports for the air-concentration meter is shown in 3.

Some of the experimental runs also required taking velocity traverses in icals through the aerated flows. The instrumentation for making these ervations has been previously described(8) in some detail. It consists entially of a device for timing the travel of minute salt-water cloudlets in-ed into the air-water mixture. The operating principle of the velocity er consists basically of marking a small element of flowing mixture and recording the time interval required for this marked element to traverse ked distance. The marking is accomplished by making a diminutive portion owing water more highly conductive to electrical current by injecting a ll amount of salt solution into the stream. The passage of this ionized dlet is then detected by electrodes at stations fixed 3 in. apart in the flow . The injections were made at a speed of about 20 per sec with individual ctions of about 0.03 cu cm of 6-per cent saline solution injected in about 0 sec. The accuracy of the air-concentration meter and the velocity er was checked by comparing the measured water rate entering the

channel with the integrated flow determined by the velocity traverses and the air-concentration traverses over the channel cross section.

Experimental Data

For the air-distribution measurements described in this report, the channel slope and water discharge were varied independently. Experiments were made for slopes of 7.5, 15, 22.5, 30, 37.5, 45, 60, and 75 degrees. For each slope, measurements of air concentration were made for total-water discharges from 2.2 to 9.6 cfs and for some slopes up to 15.0 cfs.

The measurements were made at Station 45 (45 ft from the inlet) in a ver cal centerline plane. Readings were taken at 0.01-ft intervals on a line normal to the channel bottom with the lowest point 0.02 ft from the bed and the uppermost point where the probes were completely out of the flow and th meter registered zero water concentration. The concentration profile was obtained for each slope and each water discharge after the channel had been adjusted for equilibrium conditions. For this purpose equilibrium flow was considered to be that condition of the flow of the air-water mixture for whicl the air-concentration distribution was the same at two sections 10 ft apart along the channel. This condition could be obtained by the adjustment of the initial depth of flow through control of the inlet-gate opening. Repeated measurements of the profiles were made at the two sections for different ga openings until the two air-concentration profiles were similar. Comparison were made of the concentration at corresponding distances normal to the be A typical comparison of the profiles at the two stations for equilibrium flow is shown in Fig. 4.

The results of the experiments in the form of measured concentrations fo each slope and discharge have been tabulated in Appendix B for reference. tabulation of computed parameters is given in Appendix C.

The air concentration C is defined as the ratio of the volume of air per unit volume of air and water as measured by the concentration-meter probe It is assumed that the concentration as measured represents the average value of the concentration in the region described by the probes of the conce tration meter and is applicable to the midpoint of the probes. Essentially continuous curves of air concentration with respect to distance normal to th bed were obtained for each experiment. The concentration increases gradua ly from the bed—more rapidly in the central portion and then more slowly ir the upper region, apparently asymptotically approaching the limit of 100 per cent air concentration, as is shown in Figs. 4, 8, and 9. It appears that the complete curve is comprised of two parts which have basically different characteristics. Although there is no sharp demarcation between the two parts, the curves tend to support the description that in the lower regions of the stream air bubbles are suspended in water, while the upper regions con-sist of water droplets in air. The existence of these water droplets can rea ly be detected by holding one's hand just above the main flow.

Since there is no definite upper boundary to the air-entrained flow in an open channel, questions of the definition of the linear dimensions of the flow arise. A number of different depth parameters related to the air concentrat and distribution may be defined, each having particular applications in de-scribing the bulk-flow conditions. These depth parameters may be describe as follows:

(1) A depth $\bar{d}$ is used to represent a mean depth of flow that would exist if all of the entrained air were removed up to the highest point where water is found. It therefore corresponds to the depth of a nonaerated flow of a given discharge with a velocity equal to that of the entrained flow. It may be defined as

$$\bar{d} = \int_0^{\infty} (1 - C)\,dy \tag{1}$$

where y is the normal distance from the bed and C is the concentration of air as a function of y. The value of $\bar{d}$ can be approximated with sufficient accuracy by a summation of the products of the water concentrations and their corresponding incremental depths.

(2) An upper boundary of the air-entrained flow or an upper depth d_u may be defined as the value of y where the air concentration has some arbitrarily prescribed value. For some purposes, such as determining the mean depth, and because of the asymptotic nature of the curve, it is convenient to define d_u as that value of y where the air concentration C is 0.99. Other depths may be similarly defined. Such a depth has previously been defined (4) as that corresponding to that value of y where the air concentration is C = 0.95. This depth was chosen arbitrarily as a depth where the concentration of air can be reliably measured and which includes from 98 to 99 per cent of all of the water and consequently represents a maximum distance from the bed where water will be found.

(3) A third depth parameter may be defined on the basis of the air-concentration distribution as that depth d_T which represents the value of y where the transition from the distribution in the lower regions to that in the upper regions occurs. Its value would then depend upon the characteristics of the channel as well as the air distribution. This depth parameter will be discussed in detail in a later section.

The mean air concentration in the vertical can also be defined in different ys depending upon the particular application. The mean concentration over e whole range of concentrations measured is defined as

$$\bar{C} = \frac{1}{d_u} \int_0^{d_u} C\,dy \tag{2}$$

ere $\bar{C}$ is the mean concentration of the entire flow and d_u is the so-called per limit where the concentration is 0.99. Another mean concentration $\bar{C}_T$ y be defined as the mean concentration in the region below the transition pth y = d_T which applies to that air which is being transported by the flow, t is

$$\bar{C}_T = \frac{1}{d_T} \int_0^{d_T} C\,dy \tag{3}$$

Dimensional plots of $\bar{d}$, d_u, and d_T for the various discharges and for each slope are presented in Fig. 5 to indicate the range of values as determined from the concentration data. In Fig. 5(c) the mean depth $\bar{d}$ varies with discharge and slope in much the same manner as nonaerated flow. The mean depth increases regularly with the discharge and decreases with the slope. The data for the upper limit d_u (Fig. 5(a)) indicate that for a given discharge d_u tends to decrease at first and then to increase as the slope increases; consequently, the maximum depth tends to be nearly the same for corresponding discharge rates on the various lower slopes. It appears that the increase depth due to aeration increases with both slope and discharge; however, in combination with the mean depth, which decreases with slope and increases with discharge, the upper limit tends to decrease and then increase as the slope increases. As a result, over a considerable range of slopes (15 to 30 degrees), the upper limit is essentially the same and depends only on discharge. A similar situation exists in regard to d_T (Fig. 5(b)) in that it also shows a tendency to decrease gradually and then to increase gradually as the slope increases. Consequently, the range of variation of d_T for any given discharge is relatively little over a wide range in slopes.

The mean concentration $\bar{C}$ for the entire flow and $\bar{C}_T$, the mean concentration in the region below d_T, have been plotted in Figs. 6(c) and 6(b) in terms of the discharge and for each slope. It is interesting to note that the mean concentration for the entire flow is nearly constant for a quite wide range of discharges but increases very considerably with the slope. The mean concentration $\bar{C}_T$ for the region below d_T has similar characteristics. In Fig. 6(a) the concentration C_T at the transition depth is plotted. This concentration varies but little with the depth but increases with the channel slope.

Analysis of the Data

Distribution of Air in Aerated Flow

A qualitative concept of air-entrainment phenomena has been established with the aid of the air-concentration determinations made transverse to the direction of self-aerated flows, high-speed photographs and motion pictures of the flow, and other similar information. Since it has been shown that aeration of flows on a steep slope begins in a region where the boundary layer generated by the channel bed has reached the water surface, it is to be presumed that aeration depends primarily upon the turbulence intensity of the flow. Aeration appears to occur when the transverse velocities of turbulence are sufficiently strong near the air-water interface to cause clumps of water to break through the surface into the air and then fall back by gravity into the flowing stream. This breaking away of clumps of water from the main stream and the falling back into the stream with attendant splashing and breaking into a heterogeneous spray of globules and droplets is associated with the insufflation of air into the stream. It occurs particularly with water flow at high velocities on steep slopes. The air carried back is then distributed throughout the flow by turbulent transfer. Observations by means of instruments indicate that two regions of self-aerated flow develop: (1) an upper region of heterogeneous clumps, globules, and droplets of water ejected from the flowing liquid stream into the atmosphere at more or less arbitrary velocities,

1 (2) a lower region consisting of air bubbles distributed through the flow turbulent transport fluctuations, which can be described by some boundary-rer equation. Between the two regions is a transition zone defined by a insition depth which is a fluctuating surface necessarily at a statistical ean elevation above the channel bottom.

stribution of Air in Upper Region

If it is assumed that the components normal to the bed of the turbulent ocity fluctuations are randomly distributed such that the mean value is ro, those in the outward direction will constitute one-half of a Gaussian tribution. Those in or near the surface that penetrate the surface will rry large clumps and smaller globules of water into the atmosphere against force of gravity a distance proportional to the square of the individual insverse velocities. The maximum height of the trajectories that they low will also be randomly distributed about zero. (The path will be a long, t trajectory because of the large, longitudinal component of the local ve-ity.) If the distances from the surface from which the particles are pro-ted are represented by one half of a Gaussian distribution, the frequency ') of particles projected a distance y' above the transition depth may be ex-essed as

$$f(y') = \frac{2}{h\sqrt{\pi}} e^{-\left(\frac{y'}{h}\right)^2} \tag{4}$$

ere h is a measure of the mean distance the particles are projected above . The factor 2 is used to indicate that only particles being projected in the ward direction are being considered; that is, half of the complete Gaussian being taken so that $\int_0^\infty f(y')\,dy' = 1$. Furthermore, since it is presumed t the flow is uniform on the average, particles are equally likely to be pro-ted at all points of the transitional surface. Then the proportion of all ticles leaving a unit area of the transitional surface that reach or pass ough a corresponding area at elevation y' will be

$$P_{y'} = \frac{2}{h\sqrt{\pi}} \int_{y'}^{\infty} e^{-\left(\frac{y'}{h}\right)^2} dy' \tag{5}$$

l the number of particles reaching or passing through this area in both di-tions during the sampling period will be

$$N_{y'} = N_T P_{y'} \tag{6}$$

ere N_T is the total number of particles that leave or return to the transition face per unit area during the sampling period. The average number of ticles of water that reach or pass through any horizontal area per unit e may be assumed proportional to the average concentration of water parti-s. Therefore, using Eqs. (5) and (6), the air concentration at any distance above the transition level in terms of the concentration at the transition el C_T and the representative height h is

$$\frac{1-C}{1-C_T} = \frac{2}{h\sqrt{\pi}} \int_{y'}^{\infty} e^{-\left(\frac{y'}{h}\right)^2} dy' \quad (7)$$

Eq. (7) is, of course, the cumulative Gaussian probability curve and is applicable to the aeration phenomenon only for $y' > 0$. The gradient of the concentration is from Eq. (7)

$$\frac{dC}{dy} = \frac{2(1-C_T)}{h\sqrt{\pi}} e^{-\left(\frac{y'}{h}\right)^2} \quad (8)$$

and when $y' = 0$, $\frac{dc}{dy}$ is a maximum whose value is

$$\left(\frac{dC}{dy}\right)_{max} = \frac{2(1-C_T)}{h\sqrt{\pi}} \quad (9)$$

and occurs where the concentration is C_T and the depth is d_T.

The constants C_T and h are properties of the flow related to the intensity of the turbulence in the neighborhood of d_T which, in turn, is probably dependent upon the velocity and roughness of the channel.

Distribution of Air in Lower Region

The air entrained at the transition surface by the turbulent fluctuations is then distributed throughout the flow in the region below d_T by the turbulence in the stream resulting in a statistical equilibrium between the bouyancy of the air and the concentration gradient. Such equilibrium is described by the well-known equation

$$-CV_b + \epsilon_b \frac{dC}{dy} = 0 \quad (10)$$

where C is again the air concentration at a normal distance y from the bed; V_b, the rising velocity of the air bubbles, is taken as being negative; and ϵ_b is a mixing parameter for the air-bubble transfer. In aerated flows the value of ϵ_b as a function of y is unknown, and a number of assumptions as to the form of the function can be made. However, it may be presumed that the air bubbles are transported in the same manner as momentum; and it appears reasonable as an approximation to assume ϵ_b to be proportional to ϵ_m, the momentum mixing parameter, as is done in the case of transport of other substances such as sediment. In open channels in the nonaerated condition where the shear is linearly distributed from zero at the surface to a maximum at the bed and the velocity profile is considered to be logarithmic throughout the depth of flow, it has been shown[9] that ϵ_m has a parabolic form being zero at the surface and at the bed with a maximum at the centerline and can be expressed in the form

$$\epsilon_b = \beta k \sqrt{\tau_o/\rho} \left(\frac{d_T - y}{d_T}\right) y \quad (11)$$

re $\sqrt{\tau_o/\rho} = V_*$, the shear velocity, and τ_o is the boundary shear at the , ρ is the density of the fluid, β is a proportionality factor in $\epsilon_b = \beta\epsilon_m$, k is the Karman universal constant. In the case of transport in a closed nnel, however, the shear at the upper boundary is not zero and the maxi- m velocity occurs at a point between the upper and lower boundary, at ch point the shear is also zero. This requires that ϵ_m also be zero at this nt and that the distribution of ϵ_m consists of two parabolas having zero at bed, the point where the maximum velocity occurs, and at the upper ndary and maximum values at intermediate points above and below the nt of maximum velocity. It has been shown,[10] however, that suspended tter is actually transported transversely across the level of zero mixing fficient, and it has been suggested that the mixing coefficient be considered stant in this region. Since it is likely that the vertical transport of sus- ded matter depends only upon the transverse velocity fluctuations in the tical direction, such transport may well occur even if the correlation be- en the vertical and longitudinal fluctuations approaches zero.

Measurements of velocity distribution in aerated flows show[4,5] that the ocity tends to decrease as the upper limit of the flow is approached and t the maximum occurs well down in the flow proper. This implies that a ar is developed in the transition zone. A plausible explanation for such hear is the change in momentum engendered by the return of water clumps droplets to the flow at the end of their trajectory through the atmosphere ve d_T. In the course of that trajectory, they suffer a loss of velocity due tmospheric drag on the mass. The shear developed at d_T is analogous to ıpper boundary and a distribution of the type found for closed channels ht be expected.

The complex distribution found for closed channels may be approximated a parabolic distribution over the entire region below d_T, with the maximum ıe incorporated in the factor β. This approximation is equivalent to as- ning a distribution for ϵ_b as given by Eq. (11). If this value for ϵ_b, given Eq. (11), is substituted, Eq. (10) becomes

$$C V_b = \beta k \sqrt{\tau_o/\rho}\left(\frac{d_T - y}{d_T}\right) y \frac{dC}{dy} \qquad (13)$$

ch upon integration, assuming that V_b is independent of y, is

$$C = C_1 \left(\frac{y}{d_T - y}\right)^z \qquad (14)$$

which

$$z = \frac{V_b}{\beta k \sqrt{\tau_o/\rho}} = \frac{V_b}{\beta k V_*} \qquad (15)$$

C_1 is a constant whose value is the concentration at $y = \frac{d_T}{2}$.

Application to Experimental Data

The use of Eqs. (7) and (14) as a description of the air-concentration distribution characteristics depends upon the degree to which they fit the experimental data. There are two factors which have a bearing on this: the form of the equation and the magnitude of the constants. Whether or not the form of the equations is reasonably correct depends upon the extent that the assumptions made in their development approximate the actual mechanism. The constants involved depend upon the intensity of the turbulence and at the present cannot be determined analytically but must be evaluated empirically from systematic experimentation.

Eq. (9) permits the determination of the transition depth d_T and the concentration C_T at that depth. From a plot of the concentrations in terms of the normal distance from the bed, the point where the gradient $\frac{dc}{dy}$ is a maximum was graphically located, as shown in Fig. 7(a). The corresponding values of d_T and C_T are noted. Using this value of C_T, the ratio $\frac{(1-C)}{2(1-C_T)}$ was plotted as a function of y on so-called probability paper, upon which a cumulative Gaussian distribution plots as a straight line. The value of h, or rather the value of σ, the standard deviation of the distribution, wherein $h = \sigma\sqrt{2}$, is obtained from the slope of this straight line. Alternately, h can be determined directly from Eq. (9) by evaluating the gradient at the point where $\frac{dc}{dy}$ is a maximum and inserting the value of C_T obtained for that point. Fig. 7(b) shows a plot of the concentrations in the upper region of the flow plotted in this manner with the standard deviation indicated. It is apparent from this plot that the concentrations possess a cumulative Gaussian distribution.

In the region of flow below d_T, Eq. (14) indicated that the concentration C is a function of $\frac{y}{d_T - y}$ which, when plotted logarithmically, results in a straight line whose slope is equal to the exponent z and C_1 is the value of C at $\frac{y}{d_T - y} = 1$. The concentration data for the experiment have been plotted this manner in Fig. 7 to show the degree of agreement with the form of Eq. (14) and to indicate the evaluation of C_1 and z. Inasmuch as C approaches infinity as y approaches d_T, the experimental data must depart from the cu for the larger values of $\frac{y}{d_T - y}$, but in this case the data agree with the cu up to $\frac{y}{d_T} = 0.9$.

The straight lines drawn through the data in Figs. 7(b) and 7(c) have been transposed to the original plot in Fig. 7(a) to show the characteristics of th curves on the arithmetic scales. Eqs. (7) and (14) have also been plotted along with the data for a number of typical experiments using various discharges and slopes shown in Figs. 8 and 9 for purposes of comparison. Because of the character of Eq. (14) in the neighborhood of $y = d_T$, the two curves are connected by a transition to pass from one curve to the other.

Relation of Parameters to Flow Conditions

Based upon the above analysis of the aeration phenomenon, a number of parameters were developed as being descriptive of both the magnitude and

tribution of the air. These parameters varied with the flow conditions
h as the channel slope and discharge, which in turn, along with the rough-
s characteristics of the channel, govern intensity and scale of the generat-
turbulence. The flow turbulence is created initially at the bed by the wakes
l eddies formed by the flow over the roughness elements; the turbulent
lies are then diffused upward into the flow stream. The intensity of turbu-
ce at the transitional surface which appears to be paramount in the aeration
cess should then depend upon both the initial generation and the depth of
mixture to the transitional surface. The intensity of the turbulence is re-
ed to the boundary shear τ_0 on the bed, which can be expressed by a so-
led shear velocity $V_{*b} = \sqrt{\tau_0/\rho}$. Because of the shear that is presumed to
st at the transitional surface, it is not possible in the present experiments
differentiate the bed shear from the total shear. As a first approximation,
total shear or a shear velocity based upon the total boundary shear may
used as a measure of turbulence intensity. To test this hypothesis, the
an concentration was plotted in terms of $V_*/d_T^{2/3}$ where V_* is defined as

$$V_* = \sqrt{g d_T \sin\alpha} \tag{16}$$

l d_T is the transitional depth previously defined. The expression $V_*/d_T^{2/3}$
empirical and was chosen as that form which best correlated the data.
s plot is shown in Fig. 10. The measured values of the mean concentration
reasonably well on a single curve with a tendency for the points corre-
nding to the smaller discharges to depart from the mean curve. When the
an concentration in the area below d_T, $\bar{C}_T$, is considered as being the air
ch is insufflated and distributed through the flow, the correlation is even
ter, as shown in Fig. 11. For the higher discharges for each slope the
nts all fall on a smooth curve, while there is a clear departure from the
ve for the low discharges on the corresponding slopes. It appears that as
discharge decreases below a certain value for a given slope, an additional
tor becomes important in reducing the concentration. The nature of this
tor is not readily apparent, but it should be noted that a decrease in dis-
rge corresponds to a decrease in d_T, and it is quite possible that an insta-
ty in flow pattern arises when the depth becomes small. Considering the
a as a whole, it is rather significant that when plotted in this manner, mean
concentrations ranging from about 0.05 to 0.75 corresponding to discharges
ging from about 4 to 15 cfs on slopes with ranges from 7.5 to 75 degrees
correlated along a single curve.

The parameter $V_*/d_T^{2/3}$ includes the depth d_T to the transitional surface.
s depth is a function of the air-concentration distribution and is difficult
letermine explicitly. It is, however, related to the discharge and slope and
eneral to the channel roughness. In these experiments the channel rough-
s is a constant so that for these data, d_T depends only on the slope and
charge. In terms of these measured characteristics, $V_*/d_T^{2/3}$ is related
$/q^{1/5}$ where $S = \sin\alpha$ and α is the angle of inclination of the bed and q is
unit discharge of water. Consequently, the air concentration that may be
ected in a high-velocity open channel can be related to the flow charac-
stics. This relationship is shown in Fig. 12, where the mean concentration
measured is plotted as a function of $S/q^{1/5}$ for the particular channel in
ch the experiments were performed. For channels of different roughness
racteristics, the curve will probably be similar but shifted an amount to

correspond to the different roughness. A similar plot is given in Fig. 13 showing the mean concentration between the bed and the transitional surface where the depth is d_T. Here again, as in Fig. 11, correlation between $\bar{C}_T$ and $S/q^{1/5}$ appears to be better than that between $\bar{C}$ and $S/q^{1/5}$.

The results shown in Fig. 13 provide a means of estimating the mean air concentration that may be expected in an equally rough, wide open channel o known unit discharge and slope. The depth of flow or the magnitude of the various previously described depth parameters, which are important in design, is a function of the air concentration.

Effect of Entrained Air on the Flow

It was suggested above that the air insufflated into the flow is distributed throughout the flow by turbulent transport. The entrained air has the effect of changing the flow from one of water alone to the flow of a mixture of air and water. Several characteristic depths were defined, such as d_u to represent the uppermost extent of the water particles projected from the flow, $\bar{d}$ to represent the depth of flow of the water alone, and d_T to represent a new equilibrium depth brought about by the presence of air in the flow. For design purposes, it is of considerable importance to know how these paramete and the actual flow velocity are related to the bulk flow properties or what differences might be expected in these parameters in relation to a corresponding nonaerated flow for which the depth may be computed by means of well-established flow formulas such as the Manning or Chezy formula.

In order to investigate such a relationship, experiments were performed in the experimental channel to establish the resistance of the channel to non aerated flow, that is, flow of water alone. The slope was decreased so that for comparable discharges the flow would be nonaerated. These slopes ranged from approximately 1 degree to a maximum of about 6.5 degrees, above which aeration was observed to occur. The discharges were in the same range as those used in the aeration experiments. Normal flow was established for each combination of slope and discharge, and the mean depth was determined by averaging the depths measured at various points along th centerline.

The relationship between the depth of flow for the nonaerated condition a the discharge and slope is shown in Fig. 14, where the measured depth d_m i plotted as a function of $q/S^{1/2}$ on logarithmic scales. The equation of the line drawn through the points is

$$q = 90\,5\, d_m^{3/2} (\sin\alpha)^{1/2} \quad (1$$

For the range of the variables encountered in the nonaerated flow, it appear that the Chezy formula best describes the data and the coefficient 90.5 char terizes this particular roughness. The mean velocity is then

$$V_m = 90.5\, d_m^{1/2} (\sin\alpha)^{1/2} \quad (1$$

If it were assumed that the same type of flow equation applied to aerated flow, the mean velocity of the air-water mixture flowing at a depth d_T is

$$\bar{V} = C d_T^{1/2} (\sin\alpha)^{1/2} \tag{19}$$

ere V is the mean velocity of the air-water mixture and C is the Chezy efficient. Assuming that the velocity of the air and water components are ıal, then Eq. (19) can be written in terms of the water discharge, as

$$q = C \bar{d} d_T^{1/2} (\sin\alpha)^{1/2} \tag{20}$$

ere $\bar{d}$ is the mean depth computed by Eq. (1) and represents the cross-ctional area of the water. Eq. (20) was tested by plotting the experimental ;a for aerated flow in the form $\bar{d}d_T^{1/2}$ as a function of $q/(\sin\alpha)^{1/2}$ in Fig. . Superimposed upon the plot is the line for nonaerated flow in the experi-ɛtal channel taken from Fig. 14. The data cluster about the straight line · nonaerated flow, indicating that an equation similar to Eq. (20) will corre-e the data, and, in addition, when the parameters $\bar{d}$ and d_T are used, the ezy resistance coefficient is essentially the same as that for nonaerated w. Fig. 15 shows that at least for the conditions of these experiments, air-;rained flow, when considered to be a flow of a mixture, follows the same vs as for nonaerated flow when depth parameters related to the properties the mixture are used to define the linear flow dimensions. The agreement the data for aerated and nonaerated flow shown in Fig. 15 substantiate newhat the deductions made in regard to the air-entrainment process.

An evaluation of $\bar{d}$, d_T, and d_u for specific flow conditions is not possible m Fig. 15. A clearer idea of the changes in these parameters will be ob-ned if they are compared with the depth d_m computed in the usual manner, ng Eq. (17) which includes for this channel the Chezy coefficient of 90.5. s the depth at which the water would flow in this channel at the given dis-ırge and slope assuming no air entrainment. The ratios d_u/d_m, d_T/d_m, l $\bar{d}/d_m$ as functions of mean air concentration are plotted in Fig. 16(a), b), and 16(c). In all cases, of course, the ratios approach unity as the an concentration approaches zero. That is, for water flow only, = $\bar{d} = d_T = d_m$. As the mean air concentration increases, the ratios depart m unity in such a way that as $\bar{C}$ approaches, 1.0, $\bar{d}$ becomes very small, ile d_u and d_T necessarily increase.

The upper limit of the spray d_u increases very considerably with increased concentration which in turn means an increase in the turbulence intensity project water particles farther into the atmosphere. For the highest centrations measured, the value of d_u is nearly four times as large as the ıaerated depth for a corresponding discharge and slope. The transition th, as might be expected, also increases relative to the computed depth ce increasing concentration bulks the flow and makes the effective depth ater. On the other hand, the mean depth decreases with increasing concen-tion. It appears from Fig. 16 that concentration has relatively little effect d_T and $\bar{d}$ until the air concentrations are greater than about 50 per cent. · the slopes where the air concentrations were the greatest (being about 85 cent), $\bar{d}$ was reduced to about one half of d_m while d_T was nearly twice large as d_m.

The decrease in $\bar{d}$ with increase in mean air concentration implies a corre-nding increase in the mean water velocity above that computed on the basis d_m since $\bar{d}$ is the cross-sectional area of the water flow. The trend of the

mean water velocity in terms of the nonaerated velocity in this channel is shown in Fig. 17. Here again, the velocity of aerated flow is about the same as that of a corresponding nonaerated flow until the mean concentration is i the neighborhood of 50 per cent. It then increases rather rapidly.

CONCLUSIONS

1. Self-aerated, high-velocity, open-channel flow appears to consist of two distinct phases: an upper region consisting primarily of independent droplets and larger agglomerations of water that move independently of the stream proper, and a lower region in which discrete air bubbles are suspen ed in a turbulent stream and are distributed by the mechanism of turbulenc

2. In the upper region, the distribution of water-air agglomerate and dro lets can be adequately described by the Gaussian cumulative probability equation, the constants of which are properties of the flow conditions. In th lower region, the distribution of the air agrees closely with an equation for turbulent mixing based upon an approximation for the distribution of the mi ing parameter.

3. The magnitude of the entrained air concentrations apparently depends upon the intensity of turbulent fluctuations generated at the rigid bed and the depth of flow. The mean concentration could be correlated with the parame $V_*/d_T^{2/3}$ or in terms of the flow characteristics; the mean concentration $\bar{C}$ is a function of $S/q^{1/5}$, where S is the sine function of the slope angle, and is the unit discharge.

4. The maximum height of water (or spray) above the rigid bed relative a corresponding nonaerated flow increases rapidly with mean air concentration in the flow. The transition depth is greater than the depth of a corr sponding nonaerated flow because of the bulking effect of the entrained air.

5. The mean velocity of an air-entrained flow is greater than that of a corresponding nonaerated flow by an amount that increases with the air concentration and corresponds to a decrease in the mean depth.

6. When the effective depth of aerated flow is taken to be the transition depth and the mean depth is the cross-sectional area per unit width of chan the flow is described by a formula of the Chezy type involving approximatel the same roughness coefficient as applies for nonaerated flow in the same channel.

ACKNOWLEDGMENTS

The experimental results reported here were developed as part of a program of investigation on the basic mechanics of atmospheric air insufflation and entrainment by water flowing at high velocity with a free surfac The study was sponsored by the Office of Naval Research at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota. Alan S. Goodye and Jimmie F. Hayek, research assistants at the time of this phase of the t program, assisted in the collection and analysis of the data.

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LIST OF SYMBOLS

C - Concentration, ratio of volume of air to volume of air plus water.

$\bar{C}$ - Mean concentration in vertical section.

C_1 - Concentration at point $y = d_T/2$.

$\bar{C}_T$ - Mean air concentration in region between lower boundary and d_T.

C_T - Concentration at transition depth $y = d_T$.

$\bar{d}$ - A mean depth of flow defined by Eq. (1).

d_u - Upper limit of flow or value of y where $C = 0.99$.

d_T - Transition depth between upper and lower regions of flow.

d_m - Mean depth in nonaerated flow.

g - Acceleration due to gravity.

h - A mean height to which water particles are projected above d_T.

k - Von Karman universal constant for velocity distribution.

$N_{y'}$ - Number of particles reaching or passing through a horizontal are a distance y' above transition depth d_T.

N_T - Number of particles that leave or return to an area at the transit depth.

$P_{y'}$ - Proportion of all particles leaving area at transition depth that reach or pass through horizontal area at y' above transition dept

Q - Total-water discharge.

q - Water discharge per unit width of channel.

V - Local velocity.

$\bar{V}$ - Mean velocity in vertical section = $q/d(\bar{d})$.

V_* - Shear velocity = $\sqrt{g(d_T) \sin \alpha}$.

V_b - Rising velocity of air bubbles.

y - Normal distance from channel bottom.

y' - Outward normal distance above transition depth.

z - Exponent in air-concentration equation.

α - Angle of inclination of channel.

β - Constant.

ξ_b - Mixing coefficient for air bubbles in turbulent flow.

ξ_m - Mixing coefficient for momentum transfer.

ν - Kinematic viscosity of water.

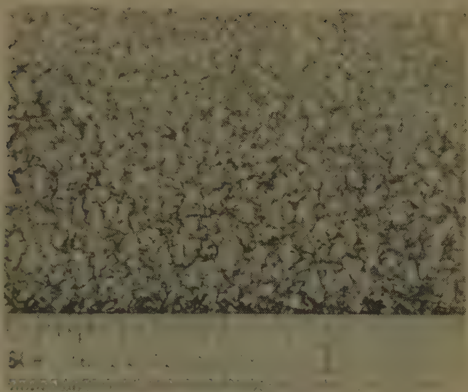
ρ - Density of water.

σ - Standard deviation of air distribution above d_T.

τ - Local shear force.

τ_0 - Boundary shear force.

ig. 1 - Variable Slope Channel for
ir-Entrainment Experiments



Smallest scale division equals 1/64 inch

Fig. 2 - Artificial Roughness Elements Installed on Channel Bed

. 3 - Probes for Air-Concentration
er

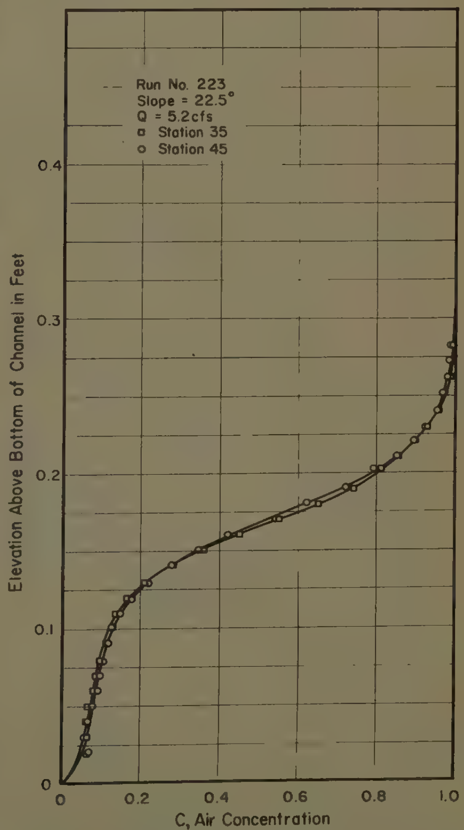


Fig. 4 - Comparison of Air-Concentration Profiles at Stations 35 ft and 45 ft From Inlet

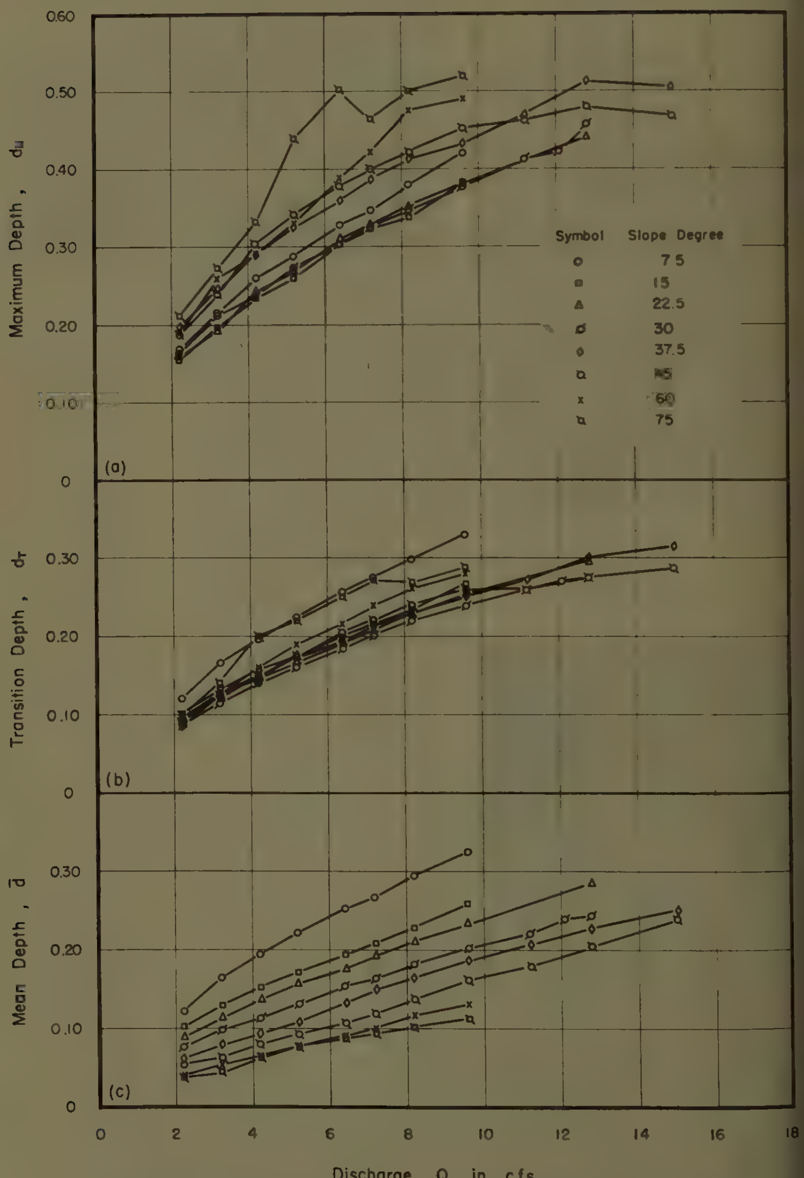


Fig. 5 - Depths of Aerated Flow as a Function of Discharge; (a) Upper Limit, d_u; (b) Transition Depth, d_T; and (c) Mean Depth, $\bar{d}$

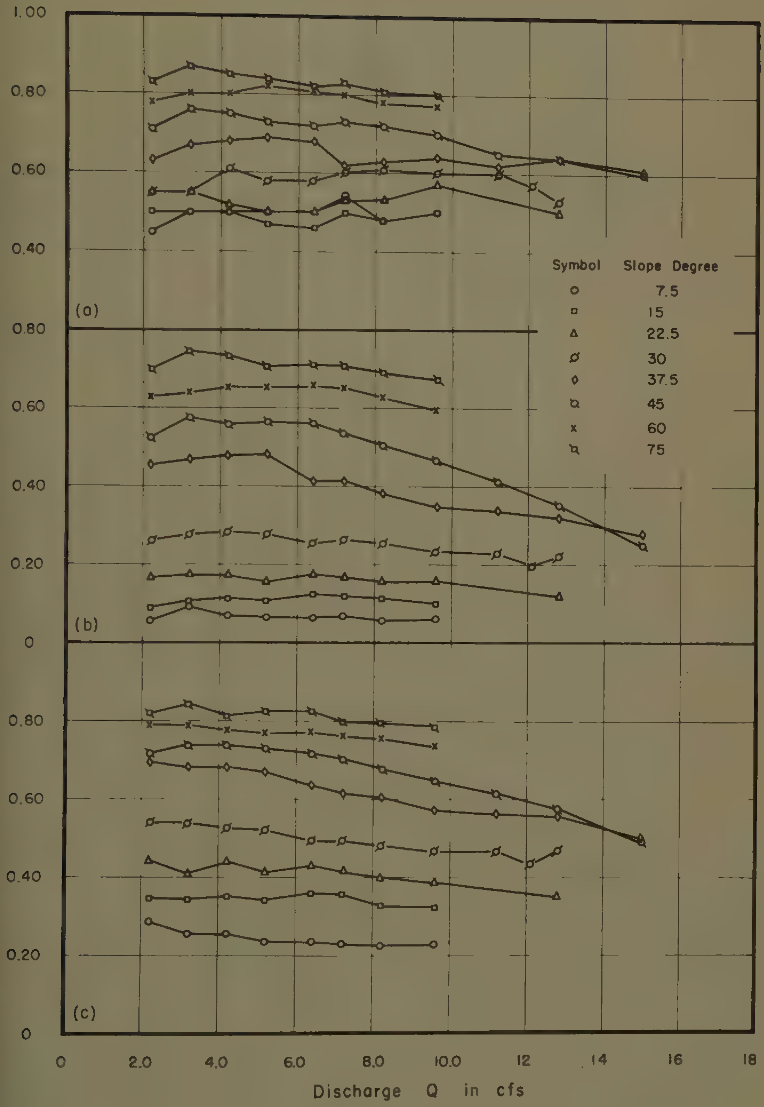


Fig. 6 - Air Concentrations as Functions of Slope and Discharge

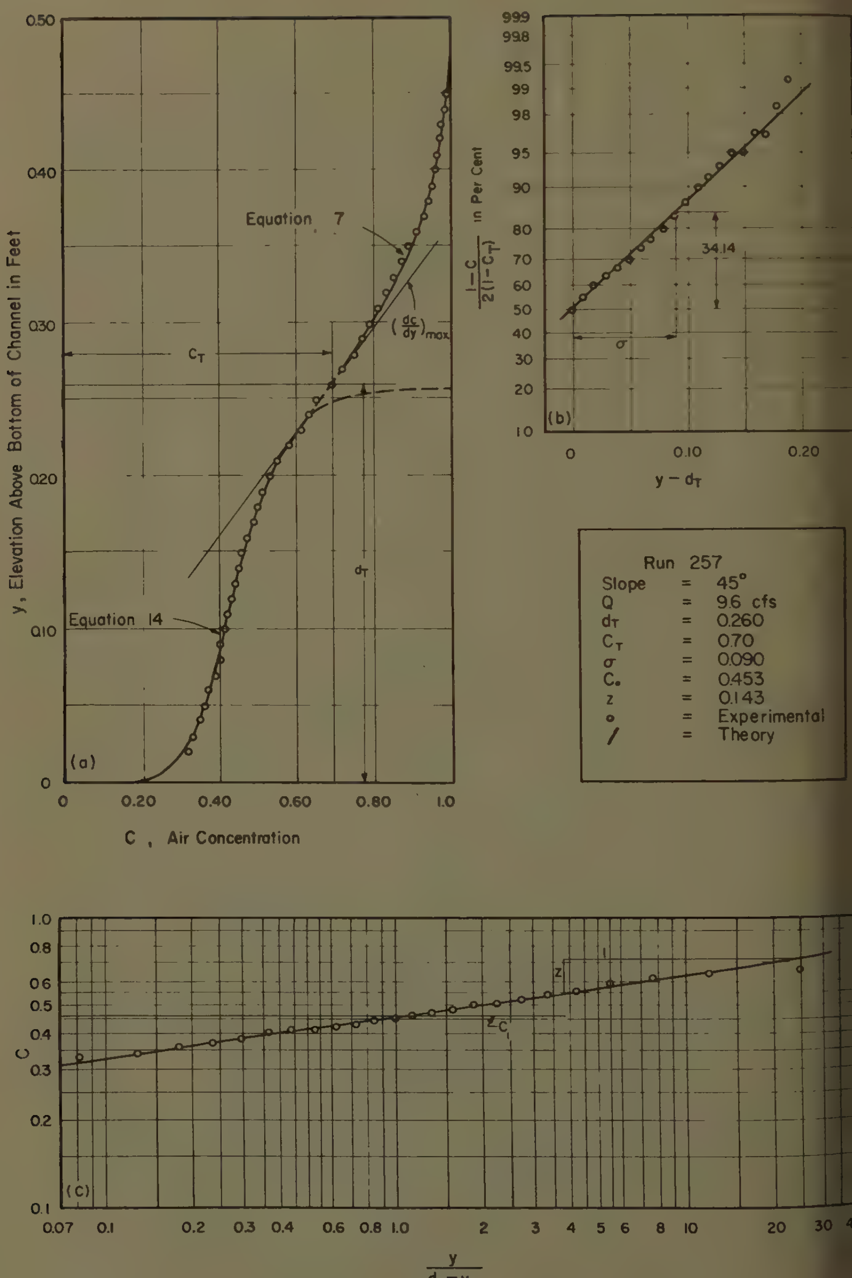


Fig. 7 - Determination of Air-Concentration Distribution Parameters; (a) Definition of Parameters; (b) Parameters in Region Above d_T; and (c) Parameters in Region Below d_T

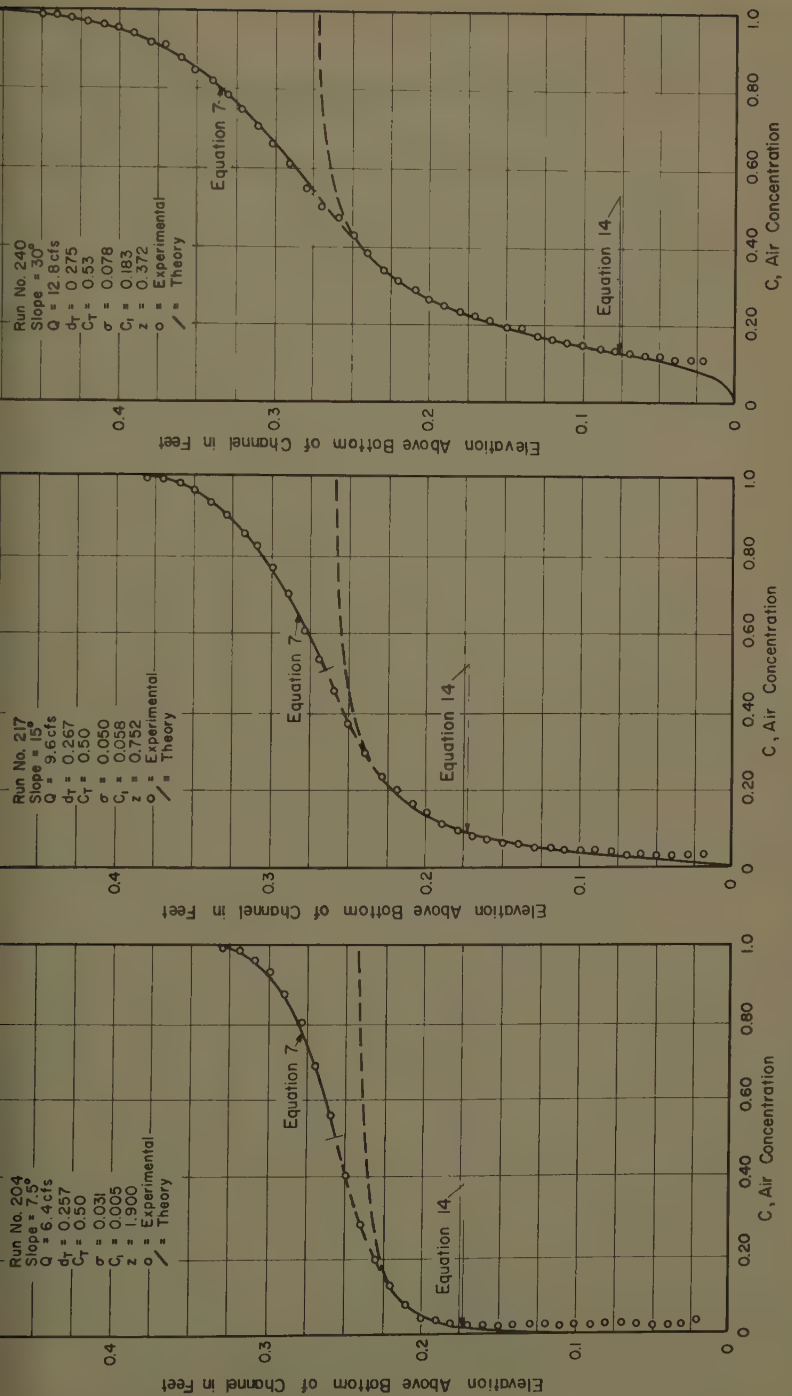
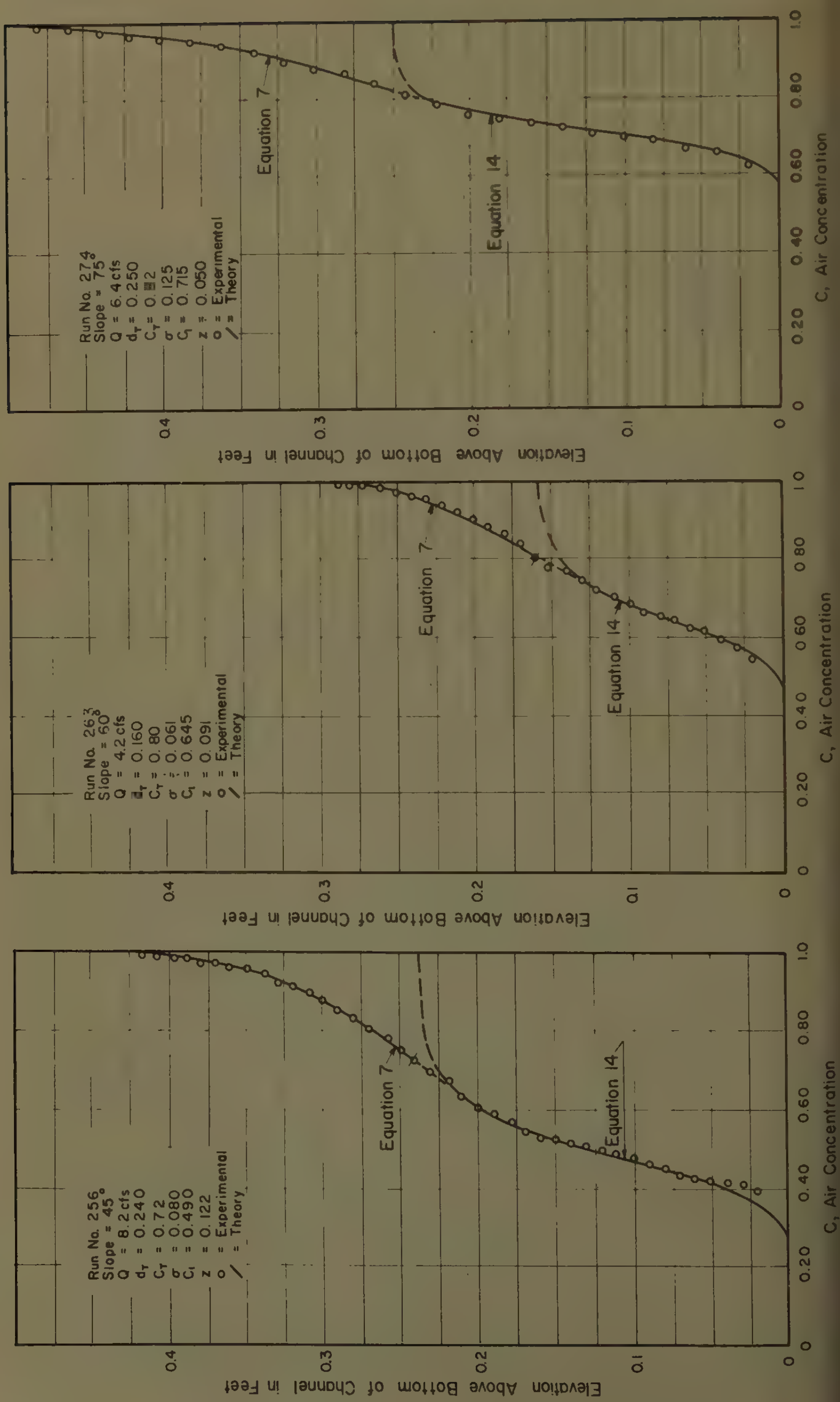


Fig. 8 – Comparison of Measured Concentrations with Theoretical Equations for Slopes of 7.5, 15, and 30 Degrees



Run No. 274
Slope = 75°
Q = 6.4 cfs
d_T = 0.250
σ = 0.125
C_1 = 0.715
z = 0.050
o = Experimental
/ = Theory
Equation 7
Equation 14
Run No. 263
Slope = 60°
Q = 4.2 cfs
d_T = 0.160
C_T = 0.80
σ = 0.061
C_1 = 0.645
z = 0.091
o = Experimental
/ = Theory
Equation 7
Equation 14
Run No. 256
Slope = 45°
Q = 8.2 cfs
d_T = 0.240
C_T = 0.72
σ = 0.080
C_1 = 0.490
z = 0.122
o = Experimental
/ = Theory
Equation 7
Equation 14
Elevation Above Bottom of Channel in Feet
C, Air Concentration
0
0.1
0.2
0.3
0.4
0.20
0.40
0.60
0.80
1.0

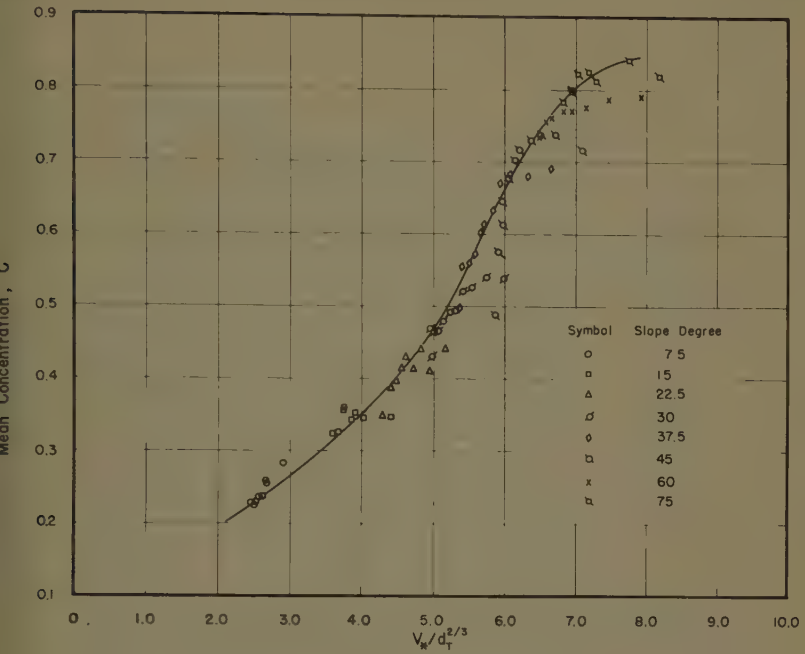


Fig. 10 - Mean Concentration for Entire Flow as a Function of $V_*/d_T^{2/3}$

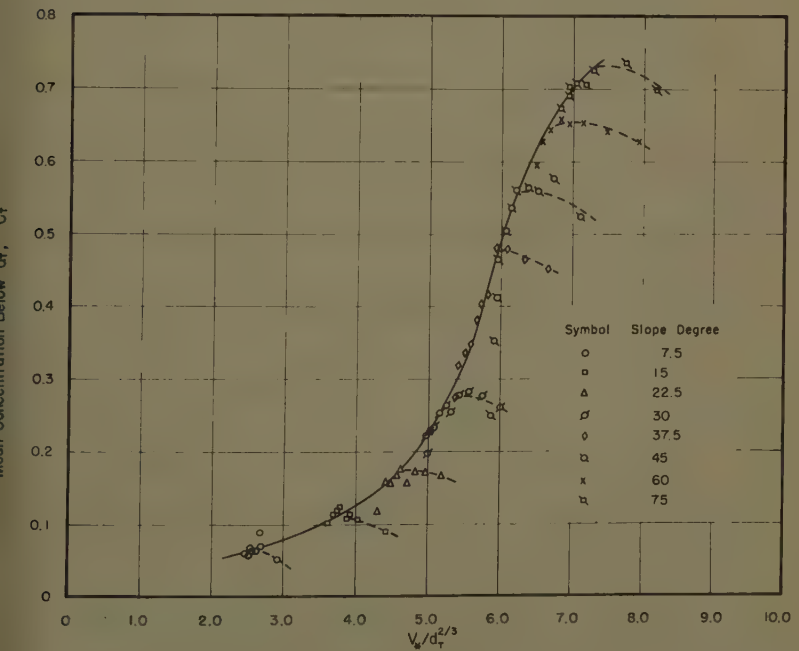


Fig. 11 - Mean Concentration in Region Below Transition Depth as a Function of $V_*/d_T^{2/3}$

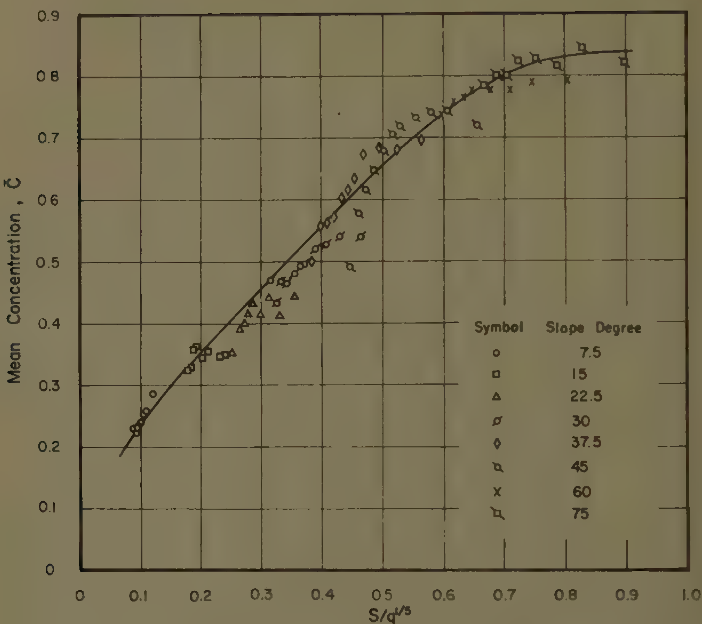


Fig. 12 - Mean Concentration for Entire Flow as a Function of $S/q^{1/5}$

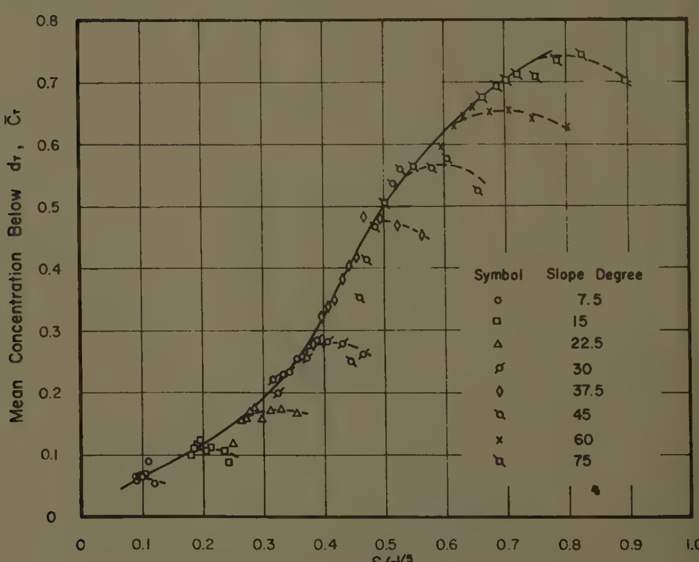


Fig. 13 - Mean Concentration in Region Below Transition Depth as a Function of $S/q^{1/5}$

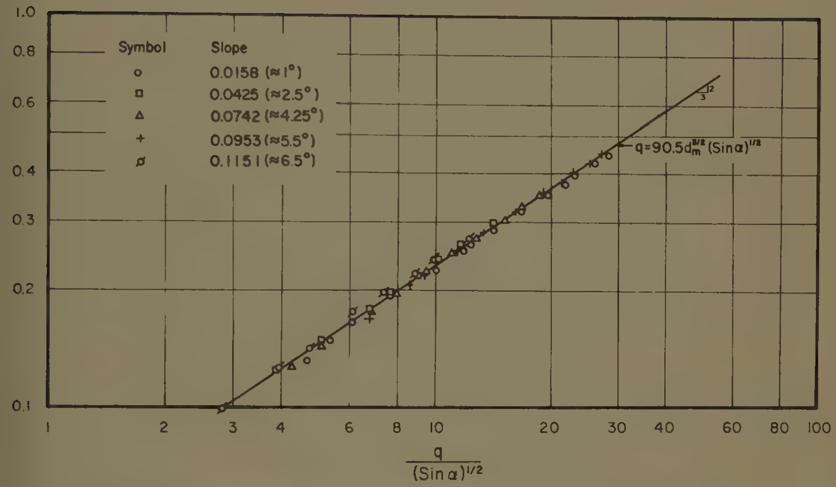


Fig. 14 - Relationship of Depth to Discharge and Slope for Nonaerated Flow in Experimental Channel

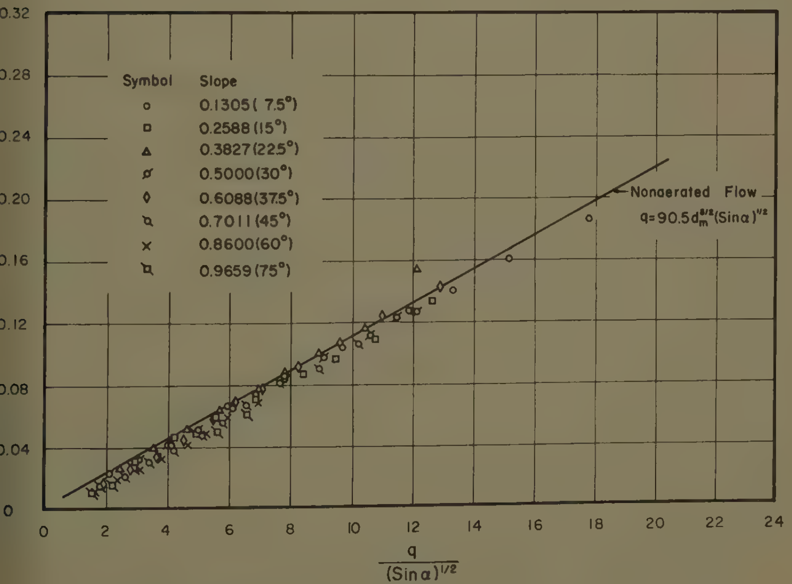


Fig. 15 - Relationship of Depth to Discharge and Slope for Aerated Flow in Experimental Channel

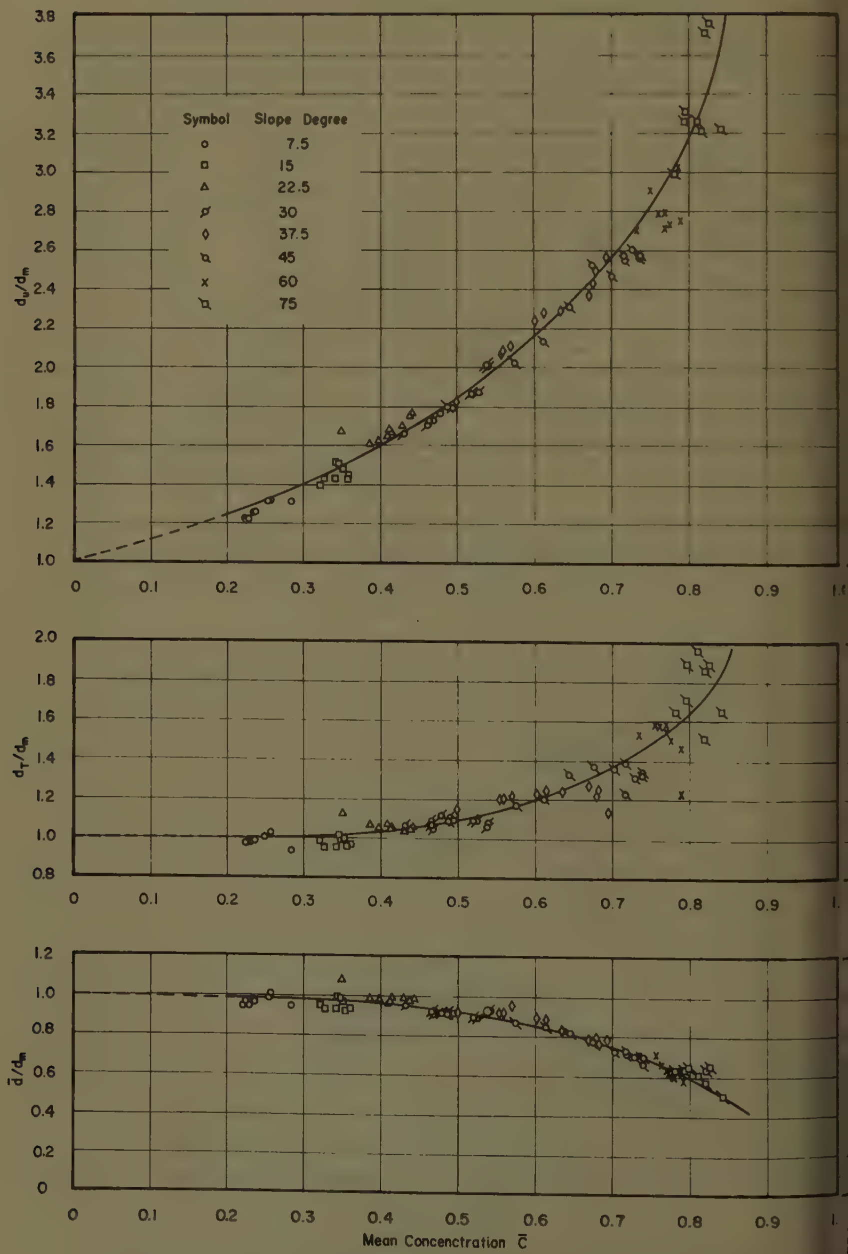


Fig. 16 - The Influence of Air Concentration on Depths of Aerated Flow Relative to Corresponding Nonaerated Flow

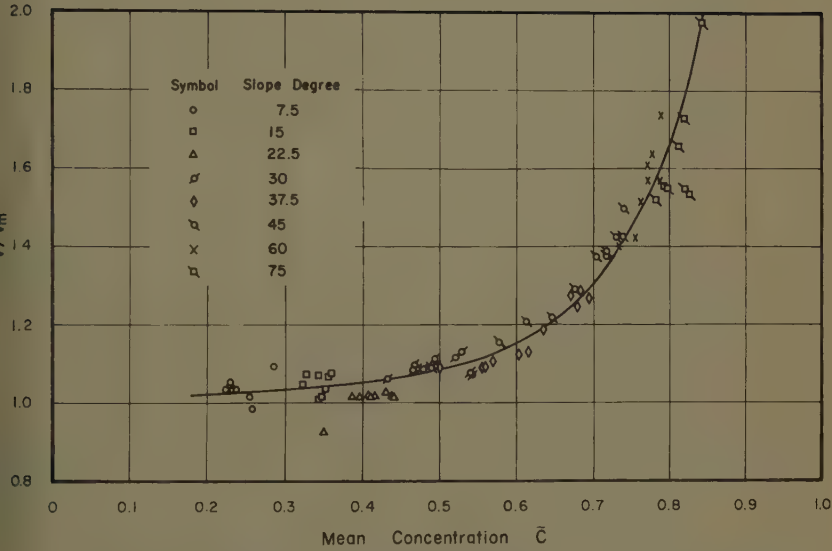


Fig. 17 - The Influence of Air Concentration on Velocity of Aerated Flow Relative to Corresponding Nonaerated Flow

APPENDIX B

(Table I - Air Concentration Distribution—For Various Discharges and Slopes)

y*	Distance Q in cfs 7.5-DEGREE SLOPE							
	2.2	3.2	4.2	5.2	6.4	7.2	8.2	9.6
0.02	0.02	0.03	0.03	0.03	0.03	0.03	0.04	0.03
0.03	0.01	0.02	0.02	0.02	0.02	0.02	0.03	0.02
0.04	0.01	0.02	0.02	0.02	0.02	0.02	0.02	0.02
0.05	0.01	0.02	0.02	0.02	0.02	0.02	0.02	0.02
0.06	0.01	0.01	0.02	0.02	0.02	0.02	0.02	0.02
0.07	0.01	0.01	0.02	0.02	0.02	0.02	0.02	0.02
0.08	0.03	0.01	0.02	0.02	0.02	0.02	0.02	0.02
0.09	0.06	0.01	0.02	0.02	0.02	0.02	0.02	0.02
0.10	0.13	0.01	0.02	0.02	0.02	0.02	0.02	0.02
0.11	0.24	0.02	0.02	0.02	0.02	0.02	0.02	0.02
0.12	0.45	0.02	0.02	0.02	0.02	0.02	0.02	0.02
0.13	0.67	0.05	0.02	0.02	0.02	0.02	0.02	0.02
0.14	0.83	0.11	0.02	0.02	0.02	0.02	0.02	0.02
0.15	0.92	0.22	0.06	0.02	0.02	0.02	0.02	0.02
0.16	0.97	0.36	0.12	0.03	0.02	0.02	0.02	0.02
0.17	0.99	0.59	0.17	0.03	0.02	0.02	0.02	0.02
0.18		0.77	0.21	0.06	0.02	0.02	0.02	0.02
0.19		0.87	0.36	0.11	0.03	0.02	0.02	0.02
0.20		0.94	0.54	0.19	0.04	0.03	0.02	0.02
0.21		0.97	0.68	0.31	0.07	0.04	0.03	0.02
0.22		0.98	0.83	0.42	0.12	0.06	0.03	0.02
0.23		0.99	0.89	0.60	0.19	0.10	0.03	0.02
0.24			0.95	0.73	0.28	0.15	0.04	0.03
0.25			0.99	0.82	0.41	0.24	0.07	0.03
0.26			0.995	0.88	0.56	0.35	0.14	0.04
0.27				0.95	0.69	0.49	0.20	0.06
0.28				0.98	0.80	0.62	0.27	0.09
0.29				0.995	0.87	0.73	0.37	0.13
0.30					0.93	0.81	0.51	0.21
0.31					0.96	0.88	0.62	0.29
0.32					0.98	0.93	0.73	0.40
0.33					0.995	0.96	0.82	0.50
0.34						0.98	0.88	0.61
0.35						0.99	0.93	0.72
0.36						0.995	0.96	0.80
0.37							0.98	0.86
0.38							0.99	0.89
0.39							0.995	0.93
0.40								0.96
0.41								0.98
0.42								0.99

y*	Discharge Q in cfs 22.5-DEGREE SLOPE										
	2.2	3.2	4.2	5.2	6.4	7.2	8.2	9.6	12.8		
0.02	0.06	0.07	0.08	0.07	0.08	0.07	0.06	0.05	0.04		
0.03	0.07	0.07	0.09	0.06	0.08	0.06	0.06	0.05	0.03		
0.04	0.08	0.08	0.09	0.07	0.08	0.07	0.06	0.05	0.03		
0.05	0.11	0.09	0.10	0.08	0.09	0.07	0.06	0.05	0.03		
0.06	0.16	0.11	0.11	0.09	0.11	0.08	0.07	0.05	0.03		
0.07	0.22	0.13	0.12	0.10	0.12	0.09	0.07	0.06	0.04		
0.08	0.34	0.16	0.15	0.11	0.13	0.10	0.08	0.06	0.04		
0.09	0.47	0.24	0.18	0.12	0.13	0.11	0.09	0.07	0.04		
0.10	0.62	0.29	0.22	0.13	0.14	0.11	0.09	0.07	0.04		
0.11	0.75	0.38	0.28	0.15	0.15	0.13	0.10	0.08	0.05		
0.12	0.84	0.49	0.37	0.18	0.17	0.14	0.11	0.09	0.05		
0.13	0.91	0.63	0.45	0.22	0.18	0.15	0.12	0.11	0.06		
0.14	0.95	0.75	0.57	0.28	0.20	0.17	0.14	0.11	0.06		
0.15	0.98	0.83	0.66	0.35	0.25	0.21	0.15	0.13	0.07		
0.16	0.99	0.89	0.75	0.42	0.30	0.23	0.17	0.15	0.07		
0.17		0.94	0.81	0.55	0.35	0.27	0.19	0.16	0.08		
0.18		0.97	0.87	0.62	0.40	0.32	0.23	0.19	0.09		
0.19		0.99	0.90	0.72	0.50	0.39	0.26	0.22	0.10		
0.20		0.995	0.94	0.79	0.60	0.46	0.35	0.25	0.12		
0.21			0.96	0.85	0.67	0.55	0.41	0.29	0.14		
0.22			0.98	0.89	0.73	0.64	0.46	0.33	0.15		
0.23			0.99	0.93	0.80	0.70	0.55	0.40	0.17		
0.24				0.95	0.85	0.77	0.64	0.47	0.18		
0.25				0.97	0.87	0.81	0.70	0.54	0.24		
0.26				0.98	0.92	0.86	0.76	0.61	0.27		
0.27				0.99	0.95	0.89	0.82	0.67	0.34		
0.28				0.995	0.96	0.91	0.85	0.74	0.39		
0.29					0.97	0.94	0.88	0.79	0.45		
0.30					0.98	0.96	0.90	0.83	0.53		
0.31					0.99	0.97	0.93	0.87	0.60		
0.32						0.97	0.95	0.90	0.65		
0.33						0.98	0.97	0.93	0.71		
0.34						0.99	0.98	0.95	0.78		
0.35						0.995	0.99	0.95	0.82		
0.36						0.995	0.995	0.95	0.85		
0.37								0.97	0.88		
0.38								0.97	0.91		
0.39								0.98	0.93		
0.40								0.99	0.94		
0.41								0.995	0.96		
0.42								0.995	0.97		
0.43									0.98		
0.44									0.99		
0.45									0.99		
0.46									0.995		

y* = Distance from channel bed (ft)

y*	Discharge Q in cfs 2.2	3.2	4.2	5.2	6.4	7.2	8.2	15-DEGREE SLOPE 9.6
0.02	0.03	0.03	0.04	0.04	0.05	0.04	0.04	0.03
0.03	0.02	0.03	0.04	0.04	0.05	0.04	0.04	0.03
0.04	0.02	0.03	0.04	0.04	0.05	0.04	0.04	0.03
0.05	0.03	0.04	0.04	0.05	0.06	0.04	0.04	0.03
0.06	0.04	0.04	0.05	0.05	0.06	0.04	0.04	0.03
0.07	0.09	0.05	0.05	0.06	0.06	0.05	0.04	0.03
0.08	0.16	0.07	0.06	0.06	0.07	0.05	0.05	0.04
0.09	0.28	0.10	0.07	0.07	0.08	0.06	0.05	0.04
0.10	0.45	0.13	0.09	0.08	0.09	0.07	0.06	0.04
0.11	0.64	0.21	0.11	0.09	0.09	0.08	0.06	0.04
0.12	0.80	0.32	0.15	0.11	0.10	0.08	0.07	0.05
0.13	0.88	0.46	0.21	0.13	0.11	0.09	0.07	0.05
0.14	0.94	0.60	0.30	0.16	0.13	0.10	0.08	0.06
0.15	0.98	0.76	0.40	0.23	0.15	0.12	0.09	0.06
0.16	0.995	0.85	0.54	0.30	0.17	0.14	0.11	0.07
0.17		0.91	0.67	0.42	0.23	0.16	0.13	0.08
0.18		0.96	0.78	0.54	0.28	0.20	0.15	0.09
0.19		0.98	0.85	0.65	0.36	0.25	0.17	0.11
0.20		0.99	0.91	0.74	0.46	0.33	0.23	0.14
0.21			0.95	0.83	0.57	0.44	0.27	0.16
0.22			0.97	0.88	0.65	0.53	0.35	0.20
0.23			0.98	0.93	0.73	0.62	0.43	0.23
0.24			0.99	0.96	0.81	0.71	0.54	0.29
0.25				0.98	0.87	0.79	0.61	0.36
0.26				0.99	0.90	0.84	0.71	0.45
0.27				0.995	0.94	0.88	0.77	0.53
0.28					0.96	0.92	0.82	0.60
0.29					0.98	0.94	0.87	0.70
0.30					0.99	0.96	0.90	0.76
0.31					0.995	0.975	0.93	0.82
0.32						0.985	0.96	0.85
0.33						0.995	0.98	0.90
0.34							0.99	0.93
0.35							0.995	0.95
0.36								0.97
0.37								0.98
0.38								0.99

y* = Distance from channel bed (ft)

y*	Discharge Q in cfs 2.2	3.2	4.2	5.2	6.4	7.2	8.2	9.6	11.2	12.1	30-DEGREE SLOPE 12.8
0.02	0.16	0.18	0.17	0.17	0.14	0.15	0.13	0.11	0.11	0.09	0.11
0.03	0.18	0.19	0.17	0.17	0.14	0.15	0.14	0.11	0.11	0.09	0.11
0.04	0.20	0.20	0.18	0.18	0.15	0.16	0.14	0.11	0.11	0.09	0.11
0.05	0.24	0.22	0.20	0.20	0.16	0.17	0.15	0.12	0.12	0.09	0.12
0.06	0.30	0.25	0.21	0.21	0.17	0.17	0.15	0.13	0.12	0.10	0.12
0.07	0.37	0.27	0.24	0.22	0.18	0.19	0.16	0.13	0.13	0.10	0.13
0.08	0.48	0.30	0.26	0.24	0.19	0.20	0.17	0.14	0.13	0.11	0.13
0.09	0.59	0.36	0.30	0.25	0.21	0.21	0.18	0.15	0.14	0.11	0.14
0.10	0.70	0.43	0.34	0.28	0.23	0.22	0.19	0.16	0.15	0.12	0.15
0.11	0.79	0.51	0.40	0.30	0.25	0.23	0.20	0.17	0.16	0.13	0.15
0.12	0.86	0.59	0.46	0.35	0.27	0.25	0.21	0.18	0.17	0.14	0.16
0.13	0.91	0.67	0.54	0.40	0.30	0.27	0.23	0.20	0.18	0.15	0.17
0.14	0.94	0.75	0.61	0.45	0.33	0.29	0.25	0.21	0.18	0.16	0.19
0.15	0.97	0.82	0.69	0.51	0.38	0.33	0.27	0.23	0.20	0.16	0.19
0.16	0.985	0.87	0.75	0.58	0.43	0.37	0.30	0.25	0.21	0.17	0.21
0.17		0.91	0.82	0.66	0.49	0.41	0.35	0.27	0.24	0.19	0.22
0.18		0.95	0.86	0.74	0.56	0.46	0.40	0.31	0.26	0.21	0.23
0.19		0.97	0.90	0.79	0.63	0.54	0.44	0.35	0.29	0.23	0.25
0.20		0.99	0.94	0.83	0.70	0.59	0.49	0.39	0.31	0.25	0.26
0.21		0.995	0.97	0.87	0.75	0.65	0.55	0.44	0.35	0.28	0.29
0.22			0.99	0.90	0.80	0.72	0.61	0.49	0.40	0.30	0.31
0.23			0.995	0.94	0.85	0.77	0.67	0.57	0.45	0.34	0.64
0.24				0.96	0.89	0.81	0.73	0.61	0.48	0.41	0.68
0.25				0.98	0.92	0.85	0.78	0.67	0.54	0.45	0.43
0.26				0.99	0.94	0.89	0.82	0.73	0.60	0.50	0.47
0.27					0.96	0.92	0.86	0.77	0.65	0.54	0.50
0.28					0.98	0.94	0.89	0.81	0.71	0.63	0.45
0.29					0.99	0.96	0.92	0.85	0.75	0.67	0.61
0.30						0.98	0.94	0.88	0.79	0.71	0.66
0.31						0.99	0.96	0.90	0.84	0.77	0.71
0.32						0.995	0.97	0.93	0.86	0.81	0.75
0.33							0.98	0.95	0.89	0.85	0.79
0.34							0.99	0.97	0.91	0.87	0.82
0.35							0.995	0.98	0.93	0.90	0.85
0.36								0.99	0.95	0.92	0.88
0.37								0.995	0.96	0.94	0.90
0.38									0.96	0.96	0.92
0.39									0.98	0.96	0.94
0.40									0.99	0.98	0.95
0.41									0.995	0.98	0.96
0.42										0.99	0.97
0.43										0.995	0.98
0.44											0.99
0.45											0.995

y* = Distance from channel bed (ft)

Table I (Continued)

y*	Discharge Q in cfs 37.5-DEGREE SLOPE										
	2.2	3.2	4.2	5.2	6.4	7.2	8.2	9.6	11.2	12.8	15.0
0.02	0.36	0.36	0.37	0.36	0.29	0.29	0.25	0.19	0.19	0.16	0.14
0.03	0.39	0.38	0.40	0.38	0.30	0.30	0.27	0.20	0.20	0.17	0.14
0.04	0.43	0.41	0.41	0.40	0.32	0.31	0.28	0.21	0.21	0.19	0.15
0.05	0.46	0.43	0.42	0.42	0.33	0.33	0.29	0.23	0.22	0.20	0.15
0.06	0.50	0.45	0.44	0.43	0.34	0.34	0.30	0.24	0.23	0.21	0.15
0.07	0.55	0.48	0.46	0.45	0.35	0.35	0.31	0.25	0.24	0.22	0.16
0.08	0.60	0.50	0.48	0.46	0.37	0.36	0.32	0.25	0.25	0.23	0.17
0.09	0.67	0.54	0.50	0.47	0.38	0.37	0.33	0.28	0.26	0.23	0.18
0.10	0.73	0.58	0.52	0.49	0.41	0.38	0.34	0.29	0.28	0.24	0.19
0.11	0.79	0.62	0.54	0.51	0.42	0.40	0.35	0.30	0.28	0.25	0.20
0.12	0.83	0.67	0.57	0.53	0.43	0.41	0.37	0.31	0.29	0.26	0.21
0.13	0.87	0.72	0.60	0.55	0.45	0.42	0.39	0.33	0.31	0.27	0.22
0.14	0.90	0.77	0.64	0.58	0.47	0.43	0.40	0.34	0.32	0.28	0.23
0.15	0.93	0.81	0.69	0.60	0.50	0.45	0.42	0.36	0.33	0.29	0.24
0.16	0.96	0.84	0.73	0.63	0.54	0.46	0.43	0.38	0.34	0.30	0.25
0.17	0.98	0.87	0.77	0.67	0.57	0.48	0.45	0.40	0.35	0.31	0.26
0.18	0.98	0.90	0.81	0.71	0.60	0.52	0.47	0.41	0.36	0.33	0.27
0.19	0.99	0.93	0.84	0.74	0.65	0.55	0.50	0.43	0.37	0.35	0.28
0.20	0.995	0.95	0.87	0.77	0.70	0.58	0.53	0.45	0.39	0.36	0.29
0.21		0.97	0.90	0.81	0.73	0.61	0.56	0.48	0.41	0.38	0.30
0.22		0.98	0.92	0.84	0.76	0.65	0.60	0.51	0.45	0.40	0.32
0.23		0.98	0.94	0.86	0.80	0.69	0.64	0.55	0.47	0.42	0.34
0.24		0.99	0.95	0.89	0.84	0.73	0.68	0.59	0.50	0.45	0.36
0.25			0.97	0.91	0.86	0.76	0.72	0.64	0.53	0.47	0.38
0.26			0.98	0.93	0.88	0.79	0.75	0.68	0.57	0.50	0.40
0.27			0.99	0.95	0.90	0.82	0.78	0.71	0.63	0.53	0.43
0.28			0.995	0.96	0.92	0.85	0.81	0.75	0.65	0.56	0.46
0.29				0.97	0.94	0.87	0.84	--	0.69	0.60	0.49
0.30				0.98	0.95	0.90	0.87	0.81	0.73	0.64	0.55
0.31				0.99	0.96	0.92	0.89	--	0.76	0.68	0.59
0.32				0.99	0.97	0.94	0.91	0.86	0.79	0.71	0.64
0.33					0.98	0.95	0.93	--	--	0.74	0.68
0.34					0.99	0.965	0.94	0.90	0.83	0.77	0.71
0.35					0.995	0.975	0.95	--	--	0.80	0.75
0.36						0.985	0.96	0.94	0.88	0.82	0.79
0.37						0.99	0.97	--	--	0.84	0.82
0.38						0.995	0.98	0.96	0.91	0.86	0.84
0.39							0.98	--	--	--	--
0.40							0.99	0.98	0.94	0.90	0.88
0.41								--	--	--	--
0.42								0.99	0.97	0.94	0.91
0.43									--	--	--
0.44									0.98	0.96	0.95
0.45									--	--	--
0.46									0.99	0.98	0.98
0.47									--	--	--
0.48									0.995	0.99	0.99

y* = Distance from channel bed (ft)

y*	Discharge Q in cfs 60-DEGREE SLOPE							
	2.2	3.2	4.2	5.2	6.4	7.2	8.2	9.6
0.02	0.54	0.53	0.54	0.53	0.54	0.52	0.50	0.47
0.03	0.58	0.56	0.57	0.55	0.57	0.54	0.51	0.49
0.04	0.61	0.59	0.59	0.57	0.58	0.55	0.53	0.50
0.05	0.65	0.61	0.61	0.59	0.59	0.56	0.54	0.51
0.06	0.68	0.63	0.62	0.60	0.60	0.58	0.55	0.52
0.07	0.72	0.65	0.64	0.62	0.62	0.59	0.56	0.53
0.08	0.76	0.67	0.65	0.63	0.63	0.60	0.57	0.54
0.09	0.81	0.69	0.66	0.64	0.64	0.61	0.58	0.55
0.10	0.84	0.72	0.68	0.65	0.65	0.62	0.59	0.56
0.11	0.87	0.75	0.70	0.66	0.66	0.63	0.59	0.56
0.12	0.90	0.78	0.72	0.68	0.67	0.64	0.60	0.57
0.13	0.92	0.81	0.74	0.70	0.68	0.65	0.61	0.58
0.14	0.94	0.84	0.76	0.72	0.69	0.66	0.62	0.58
0.15	0.96	0.87	0.78	0.74	0.70	0.67	0.63	0.59
0.16	0.97	0.89	0.80	0.76	0.71	0.68	0.64	0.60
0.17	0.98	0.91	0.83	0.78	0.72	0.69	0.65	0.61
0.18	0.99	0.93	0.86	0.80	0.73	0.70	0.66	0.62
0.19	0.995	0.94	0.88	0.82	0.75	0.72	0.67	0.63
0.20		0.95	0.90	0.84	0.77	0.74	0.68	0.64
0.21		0.96	0.92	0.86	0.79	0.76	0.70	0.66
0.22		0.97	0.93	0.88	0.81	0.77	0.72	0.67
0.23		0.98	0.95	0.90	0.83	0.79	0.73	0.68
0.24		0.99	0.96	0.91	0.84	0.80	0.74	0.70
0.25		0.995	0.97	0.93	0.85	0.82	0.76	0.72
0.26			0.98	0.94	0.87	0.84	0.78	0.74
0.27			0.98	0.95	0.88	0.85	0.80	0.76
0.28			0.99	0.96	0.89	0.86	0.82	0.78
0.29			0.995	0.97	0.91	0.87	0.83	0.80
0.30				0.98	0.93	0.89	0.85	0.82
0.31				0.98	0.94	0.91	0.87	0.84
0.32				0.99	0.95	0.92	0.89	0.86
0.33				0.99	0.96	0.93	0.90	0.87
0.34				0.99	0.97	0.94	0.91	0.88
0.35				0.995	0.97	0.95	0.92	0.89
0.36					0.98	0.96	0.93	0.91
0.37					0.98	0.97	0.94	0.92
0.38					0.99	0.97	0.95	0.93
0.39					0.99	0.98	0.96	0.94
0.40					0.995	0.98	0.96	0.95
0.41						0.99	0.97	0.96
0.42						0.99	0.97	0.97
0.43						0.995	0.98	0.97
0.44							0.98	0.98
0.45							0.99	0.98
0.46							0.99	0.99
0.47								0.99

y* = Distance from channel bed (ft)

45-DEGREE SLOPE — Discharge Q in cfs

y*	2.2	3.2	4.2	5.2	6.4	7.2	8.2	9.6	11.2	12.8	15.0
0.02	0.43	0.48	0.45	0.46	0.45	0.42	0.40	0.33	0.28	0.22	0.10
0.03	0.46	0.50	0.48	0.49	0.47	0.44	0.41	0.34	0.30	0.22	0.11
0.04	0.50	0.51	0.50	0.51	0.49	0.44	0.42	0.36	0.31	0.23	0.12
0.05	0.53	0.53	0.51	0.52	0.51	0.45	0.42	0.37	0.32	0.24	0.13
0.06	0.56	0.55	0.52	0.53	0.52	0.47	0.43	0.38	0.33	0.25	0.14
0.07	0.61	0.58	0.53	0.54	0.53	0.50	0.44	0.40	0.34	0.27	0.15
0.08	0.66	0.61	0.55	0.55	0.53	0.51	0.45	0.41	0.35	0.28	0.16
0.09	0.71	0.64	0.57	0.56	0.54	0.52	0.46	0.41	0.36	0.29	0.16
0.10	0.76	0.67	0.59	0.57	0.55	0.54	0.48	0.42	0.37	0.30	0.17
0.11	0.81	0.70	0.62	0.58	0.56	0.51	0.49	0.43	0.37	0.30	0.18
0.12	0.85	0.74	0.65	0.60	0.57	0.53	0.50	0.44	0.38	0.31	0.19
0.13	0.88	0.78	0.68	0.62	0.58	0.54	0.51	0.45	0.39	0.32	0.20
0.14	0.92	0.82	0.71	0.64	0.59	0.55	0.51	0.46	0.39	0.33	0.21
0.15	0.95	0.85	0.74	0.66	0.61	0.57	0.52	0.47	0.41	0.34	0.22
0.16	0.96	0.88	0.78	0.69	0.63	0.59	0.53	0.48	0.42	0.35	0.23
0.17	0.98	0.91	0.81	0.72	0.65	0.60	0.54	0.50	0.43	0.36	0.24
0.18	0.99	0.93	0.84	0.75	0.67	0.62	0.56	0.51	0.45	0.37	0.26
0.19	0.995	0.95	0.87	0.77	0.68	0.65	0.58	0.52	0.47	0.39	0.28
0.20		0.96	0.89	0.80	0.72	0.68	0.60	0.54	0.49	0.41	0.29
0.21		0.98	0.91	0.83	0.74	0.71	0.63	0.56	0.51	0.43	0.31
0.22		0.98	0.93	0.85	0.77	0.73	0.66	0.59	0.53	0.45	0.33
0.23		0.99	0.94	0.87	0.79	0.76	0.69	0.62	0.56	0.47	0.36
0.24		0.995	0.95	0.89	0.82	0.79	0.72	0.64	0.59	0.50	0.40
0.25			0.96	0.91	0.85	0.82	0.75	0.66	0.62	0.54	0.44
0.26			0.97	0.93	0.87	0.84	0.78	0.70	0.65	0.58	0.48
0.27			0.98	0.94	0.89	0.86	0.80	0.73	0.68	0.62	0.52
0.28			0.99	0.95	0.90	0.88	0.83	0.76	0.71	0.66	0.56
0.29			0.995	0.96	0.92	0.90	0.85	0.78	0.73	0.70	0.63
0.30				0.97	0.94	0.92	0.87	0.80	0.75	0.73	0.66
0.31				0.98	0.95	0.93	0.89	0.82	0.79	0.76	0.71
0.32				0.98	0.96	0.94	0.91	0.84	0.82	0.79	0.74
0.33				0.99	0.97	0.95	0.92	0.86	0.84	0.82	0.78
0.34				0.99	0.98	0.96	0.94	0.88	0.87	0.84	0.80
0.35					0.98	0.97	0.95	0.90	0.88	0.86	0.82
0.36					0.99	0.98	0.96	0.92	0.90	0.88	0.85
0.37					0.995	0.99	0.97	0.94	0.92	0.90	0.87
0.38						0.99	0.97	0.95	0.94	0.92	0.91
0.39						0.995	0.98	0.96	0.95	0.93	0.93
0.40							0.98	0.97	0.96	0.94	0.94
0.41							0.99	0.97	0.97	0.95	0.95
0.42							0.995	0.98	0.98	0.96	0.96
0.43								0.98	0.98	0.96	0.97
0.44								0.99	0.99	0.97	0.97
0.45								0.995	0.99	0.98	0.98
0.46									0.99	0.98	0.98
0.47									0.995	0.99	0.99
0.48										0.99	0.99
0.49										0.995	0.995

y* = Distance from channel bed (ft)

75-DEGREE SLOPE — Discharge Q in cfs

y*	2.2	3.2	4.2	5.2	6.4	7.2	8.2	9.6
0.02	0.61	0.64	0.63	0.60	0.62	0.60	0.60	0.59
0.03	0.64	0.67	0.65					
0.04	0.67	0.70	0.67	0.64	0.66	0.64	0.63	0.61
0.05	0.69	0.72	0.69					
0.06	0.72	0.73	0.70	0.66	0.67	0.65	0.65	0.63
0.07	0.75	0.75	0.71					
0.08	0.77	0.76	0.72	0.68	0.69	0.67	0.66	0.64
0.09	0.80	0.77	0.73					
0.10	0.83	0.79	0.73	0.70	0.70	0.68	0.67	0.65
0.11	0.85	0.81	0.74					
0.12	0.87	0.84	0.75	0.71	0.71	0.69	0.68	0.66
0.13	0.89	0.85	0.76					
0.14	0.92	0.87	0.77	0.73	0.73	0.70	0.69	0.66
0.15	0.94	0.89	0.78					
0.16	0.95	0.90	0.80	0.75	0.74	0.71	0.70	0.67
0.17	0.96	0.92	0.82					
0.18	0.97	0.93	0.83	0.78	0.75	0.73	0.71	0.68
0.19	0.98	0.94	0.84					
0.20	0.98	0.95	0.85	0.82	0.76	0.75	0.72	0.69
0.21	0.99	0.96	0.87					
0.22	0.99	0.97	0.88	0.84	0.79	0.77	0.74	0.71
0.23	0.995	0.98	0.90					
0.24		0.98	0.92	0.87	0.81	0.79	0.77	0.73
0.25		0.98	0.93					
0.26		0.98	0.94	0.89	0.84	0.82	0.80	0.76
0.27		0.99	0.95					
0.28		0.99	0.96	0.91	0.86	0.84	0.82	0.79
0.29		0.99	0.97					
0.30		0.995	0.97	0.93	0.88	0.87	0.84	0.81
0.31			0.98					
0.32			0.98	0.95	0.90	0.90	0.87	0.84
0.33			0.98					
0.34			0.99	0.97	0.92	0.92	0.89	0.87
0.35			0.99					
0.36				0.98	0.94	0.94	0.92	0.89
0.37								
0.38				0.98	0.95	0.95	0.94	0.92
0.39								
0.40				0.99	0.96	0.97	0.95	0.94
0.41								
0.42				0.995	0.97	0.98	0.96	0.96
0.44					0.98	0.98	0.97	0.97
0.46					0.99	0.99	0.98	0.98
0.48					0.995	0.99	0.98	0.98
0.50						0.995	0.99	0.98
0.52						0.995	0.99	0.99
0.54								0.99
0.56								0.995

y* = Distance from channel bed (ft)

Table I (Continued)

APPENDIX C

(Table II - Summary of Experimental Measurements—Rough Channel)

Run	Slope	Temp.	Disch.	$\bar{d}$	i_u	d_T	$\bar{d}d_T^{1/2}$	$q/S^{1/2}$
200	7.5°	22.3	2.2	0.1209	0.169	0.120	0.0418	4.06
201		0.4	3.2	0.1642	0.215	0.166	0.0669	5.90
202		0	4.2	0.1937	0.260	0.197	0.0860	7.75
203		0	5.2	0.2204	0.289	0.224	0.1043	9.61
204		0	6.4	0.2515	0.329	0.257	0.1274	11.80
205		0	7.2	0.2671	0.347	0.275	0.1400	13.29
206		0	8.2	0.2943	0.380	0.298	0.1605	15.10
207		0.4	9.6	0.3247	0.421	0.330	0.1865	17.71
210	15°	21.0	2.2	0.1010	0.115	0.102	0.0322	2.88
211		0	3.2	0.1289	0.197	0.132	0.0468	4.18
212		0	4.2	0.1514	0.234	0.157	0.0600	5.50
213		0	5.2	0.1701	0.260	0.174	0.0710	6.82
214		0	6.4	0.1937	0.303	0.200	0.0866	8.37
215		0	7.2	0.2087	0.323	0.215	0.0968	9.43
216		0	8.2	0.2275	0.338	0.233	0.1097	10.73
217		0	9.6	0.2591	0.383	0.267	0.1338	12.57
220	22.5°	19.6	2.2	0.0886	0.159	0.095	0.0273	2.37
221		0.8	3.2	0.1130	0.192	0.124	0.0398	3.45
222		0	4.2	0.1358	0.243	0.145	0.0517	4.53
223		0.1	5.2	0.1572	0.268	0.166	0.0640	5.62
224		0	6.4	0.1763	0.310	0.190	0.0768	6.90
225		0.1	7.2	0.1923	0.329	0.208	0.0877	7.77
226		0.1	8.2	0.2116	0.351	0.288	0.1009	8.85
227		0.8	9.6	0.2329	0.380	0.253	0.1170	10.36
228		1.2	12.8	0.2855	0.440	0.296	0.1554	12.08
230	30.0°	21.0	2.2	0.0760	0.165	0.087	0.0224	2.07
231		1.2	3.2	0.0980	0.213	0.115	0.0332	3.01
232		0	4.2	0.1125	0.238	0.139	0.0419	3.96
233		0	5.2	0.1307	0.273	0.160	0.0522	4.91
234		1.2	6.4	0.1530	0.303	0.183	0.0655	6.03
235		0.2	7.2	0.1644	0.325	0.201	0.0736	6.80
236		0.2	8.2	0.1809	0.348	0.220	0.0848	7.72
237		1.0	9.6	0.2008	0.376	0.238	0.0980	9.05
238		0.2	11.2	0.2202	0.413	0.260	0.1121	10.55
239		0.2	12.1	0.2392	0.422	0.270	0.1241	11.41
240		1.0	12.8	0.2424	0.457	0.275	0.1271	12.05

Table II - Summary of Experimental Measurements—Rough Channel

C_T	C_1	$\bar{V}$	V_*	$V_*/d_T^{2/3}$	Re x 10^{-5}	z	σ	$\bar{V}/V_m$
0.45	0.008	12.13	0.710	2.92	5.65	1.920	0.022	1.059
0.50	0.007	12.97	0.809	2.67	4.52	1.550	0.024	0.986
0.50	0.005	14.45	0.909	2.68	5.91	2.150	0.024	1.017
0.50	0.007	15.75	0.969	2.62	7.16	1.590	0.028	1.033
0.50	0.005	16.95	1.039	2.56	9.05	1.900	0.031	1.034
0.54	0.006	17.97	1.074	2.54	10.23	1.660	0.034	1.054
0.48	0.004	18.55	1.119	2.51	11.46	1.750	0.035	1.036
0.50	0.006	19.71	1.178	2.46	13.60	0.645	0.040	1.046
0.50	0.031	14.50	0.923	4.41	5.58	1.255	0.023	1.018
0.50	0.047	16.53	1.050	4.04	4.53	0.935	0.028	1.014
0.50	0.060	18.50	1.144	3.92	6.03	0.785	0.033	1.036
0.47	0.067	20.40	1.206	3.86	7.34	0.613	0.038	1.071
0.46	0.083	22.0	1.292	3.77	9.14	0.527	0.045	1.073
0.50	0.072	23.0	1.340	3.73	10.23	0.622	0.047	1.072
0.48	0.067	24.0	1.396	3.63	11.59	0.630	0.049	1.074
0.50	0.058	24.7	1.494	3.60	13.65	0.752	0.050	1.047
0.55	0.107	16.55	1.081	5.19	5.71	0.693	0.028	1.015
0.55	0.115	18.85	1.235	4.96	4.98	0.665	0.032	1.024
0.52	0.123	20.6	1.338	4.84	6.21	0.507	0.041	1.015
0.50	0.116	22.1	1.430	4.72	7.58	0.490	0.046	1.014
0.50	0.140	24.1	1.530	4.62	9.54	0.388	0.052	1.028
0.53	0.123	24.9	1.600	4.56	10.75	0.507	0.058	1.020
0.53	0.107	25.8	1.676	4.49	12.20	0.547	0.058	1.014
0.57	0.099	27.5	1.768	4.41	14.77	0.700	0.055	1.018
0.50	0.063	29.9	1.911	4.30	19.10	0.735	0.064	0.923
0.55	0.222	19.30	1.182	6.00	6.33	0.326	0.035	1.078
0.55	0.240	21.7	1.360	5.75	5.42	0.314	0.041	1.077
0.61	0.245	24.9	1.494	5.55	7.18	0.345	0.041	1.029
0.58	0.242	26.5	1.605	5.43	8.79	0.306	0.049	1.113
0.58	0.213	27.9	1.715	5.31	11.04	0.366	0.052	1.094
0.60	0.227	29.2	1.800	5.25	12.23	0.340	0.049	1.102
0.61	0.200	30.2	1.880	5.16	13.83	0.420	0.058	1.086
0.60	0.186	31.9	1.958	5.09	16.30	0.420	0.061	1.085
0.60	0.185	33.9	2.045	5.02	18.36	0.431	0.068	1.094
0.57	0.157	33.8	2.085	5.00	18.98	0.429	0.069	1.060
0.53	0.183	35.2	2.105	4.96	20.80	0.372	0.078	1.086

Table II (Continued)

Run	Slope	Temp.	Disch.	$\bar{d}$	d_u	d_T	$\bar{d}d_T^{1/2}$	$q/S^{1/2}$	$\bar{C}$
240½	37.5°	19.6	2.2	0.0605	0.198	0.085	0.0176	1.88	0.69
241		16.8	3.2	0.0783	0.245	0.120	0.0271	2.73	0.68
242		16.8	4.2	0.0923	0.291	0.148	0.0355	3.59	0.6
243		16.8	5.2	0.1073	0.326	0.175	0.0449	4.45	0.6
244		5.0	6.4	0.1317	0.361	0.195	0.0582	5.46	0.5
245		10.6	7.2	0.1494	0.388	0.212	0.0688	6.15	0.6
246		9.3	8.2	0.1638	0.413	0.227	0.0780	7.01	0.6
247		10.5	9.6	0.1855	0.433	0.250	0.0927	8.21	0.5
248		10.3	11.2	0.2064	0.471	0.272	0.1075	9.57	0.5
249		10.3	12.8	0.2275	0.514	0.300	0.1246	10.93	0.5
250		10.3	15.0	0.2521	0.505	0.315	0.1415	12.81	0.5
250½	45°	18.6	2.2	0.0530	0.188	0.090	0.0159	1.74	0.7
251		12.9	3.2	0.0625	0.240	0.125	0.0221	2.53	0.7
252		12.8	4.2	0.0791	0.303	0.151	0.0307	3.33	0.7
253		12.8	5.2	0.0921	0.342	0.172	0.0382	4.13	0.7
254		12.9	6.4	0.1065	0.378	0.205	0.0482	5.07	0.7
255		12.0	7.2	0.1182	0.400	0.220	0.0555	5.71	0.7
256		12.1	8.2	0.1361	0.422	0.240	0.0667	6.50	0.6
257		11.9	9.6	0.1600	0.452	0.260	0.0815	7.61	0.6
258		18.5	11.2	0.1782	0.462	0.260	0.0910	8.87	0.6
259		18.5	12.8	0.2029	0.479	0.275	0.1063	10.14	0.5
260		19.8	15.0	0.2397	0.469	0.278	0.1280	11.90	0.4
261	60°	19.8	2.2	0.0397	0.190	0.085	0.0116	1.58	0.7
262		14.5	3.2	0.0540	0.260	0.126	0.0195	2.29	0.7
263		18.0	4.2	0.0648	0.290	0.160	0.0259	3.01	0.7
264		18.6	5.2	0.0759	0.331	0.190	0.0330	3.73	0.7
265		18.6	6.4	0.0887	0.389	0.220	0.0416	4.59	0.7
266		19.8	7.2	0.1003	0.423	0.240	0.0492	5.16	0.7
267		19.2	8.2	0.1163	0.476	0.260	0.0593	5.88	0.7
268		14.5	9.6	0.1299	0.491	0.280	0.0687	6.88	0.7
270	75°	19.8	2.2	0.0382	0.212	0.100	0.0121	1.49	0.8
271		22.0	3.2	0.0429	0.273	0.140	0.0161	2.17	0.8
272		22.0	4.2	0.0618	0.333	0.200	0.0275	2.85	0.8
273		21.3	5.2	0.0764	0.440	0.220	0.0358	3.53	0.8
274		21.3	6.4	0.0874	0.501	0.250	0.0457	4.35	0.[illegible]
275		20.9	7.2	0.0936	0.465	0.275	0.0490	4.88	0.7
276		20.9	8.2	0.1016	0.500	0.270	0.0328	5.56	0.7
277		20.9	9.6	0.1124	0.520	0.290	0.0606	6.51	0.7

Table II (Continued)

C_T	C_1	$\bar{V}$	V_*	$V_*/d_T^{2/3}$	Re x 10^{-5}	z	σ	$\bar{V}/V_m$
0.63	0.431	24.2	1.292	6.66	7.49	0.164	0.048	1.270
0.67	0.450	27.2	1.538	6.33	11.10	0.152	0.056	1.248
0.68	0.462	30.3	1.706	6.09	15.23	0.142	0.061	1.287
0.69	0.472	32.3	1.857	5.94	19.13	0.140	0.068	1.277
0.68	0.402	32.3	1.955	5.82	15.40	0.177	0.073	1.188
0.62	0.392	32.1	2.04	5.73	19.60	0.158	0.078	1.130
0.63	0.363	33.4	2.11	5.67	21.0	0.192	0.081	1.125
0.64	0.317	34.5	2.22	5.59	24.8	0.250	0.080	1.106
0.62	0.310	36.1	2.31	5.51	28.1	0.237	0.090	1.091
0.64	0.292	37.5	2.43	5.42	32.1	0.290	0.089	1.090
0.61	0.242	39.7	2.49	5.38	35.7	0.318	0.080	1.091
0.71	0.510	27.7	1.430	7.10	8.80	0.140	0.046	1.378
0.76	0.565	34.1	1.634	6.73	12.60	0.113	0.051	1.502
0.75	0.550	35.4	1.852	6.52	16.31	0.113	0.068	1.427
0.73	0.555	37.7	1.977	6.38	19.75	0.087	0.078	1.423
0.72	0.555	40.0	2.16	6.21	25.3	0.092	0.079	1.389
0.73	0.525	40.6	2.24	6.15	26.7	0.113	0.082	1.372
0.72	0.490	40.1	2.34	6.06	29.0	0.122	0.080	1.289
0.70	0.453	40.0	2.43	5.98	31.0	0.143	0.090	1.218
0.65	0.398	41.8	2.43	5.98	38.6	0.151	0.088	1.208
0.64	0.326	42.0	2.50	5.91	41.0	0.234	0.095	1.157
0.60	0.217	41.7	2.55	5.86	44.0	0.346	0.083	1.092
0.78	0.613	37.0	1.539	7.93	11.50	0.107	0.053	1.737
0.80	0.630	38.8	1.873	7.48	15.61	0.102	0.061	1.565
0.80	0.645	43.2	2.11	7.15	24.1	0.091	0.061	1.636
0.82	0.645	45.7	2.30	6.95	30.8	0.098	0.073	1.606
0.81	0.657	48.2	2.49	6.84	37.6	0.0806	0.090	1.570
0.80	0.640	47.8	2.58	6.68	42.1	0.086	0.095	1.513
0.78	0.610	47.0	2.69	6.59	44.0	0.087	0.098	1.424
0.77	0.590	49.3	2.79	6.50	44.2	0.093	0.102	1.401
0.83	0.690	38.4	1.767	8.18	14.32	0.093	0.060	1.730
0.87	0.750	49.7	2.09	7.75	26.7	0.089	0.067	1.976
0.85	0.730	45.4	2.50	7.30	34.6	0.063	0.084	1.654
0.84	0.700	45.5	2.62	7.18	37.7	0.067	0.100	1.535
0.82	0.715	48.9	2.79	7.03	46.1	0.050	0.125	1.547
0.83	0.701	51.3	2.915	6.93	52.3	0.0579	0.094	1.550
0.81	0.687	53.8	2.90	6.93	54.4	0.0506	0.118	1.555
0.80	0.670	56.9	3.01	6.84	61.8	0.05075	0.106	1.521

Table II (Continued)

AMERICAN SOCIETY OF CIVIL ENGINEERS

OFFICERS FOR 1959

PRESIDENT

FRANCIS S. FRIEL

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WALDO G. BOWMAN
SAMUEL B. MORRIS

Term expires October, 1960:
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LLOYD D. KNAPP

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Term expires October, 1959:
CLINTON D. HANOVER, Jr.
E. LELAND DURKEE
HOWARD F. PECKWORTH
FINLEY B. LAVERTY
WILLIAM J. HEDLEY
RANDLE B. ALEXANDER

Term expires October, 1960:
PHILIP C. RUTLEDGE
WESTON S. EVANS
TILTON E. SHELBURNE
CRAIG P. HAZELET
DONALD H. MATTERN
JOHN E. RINNE

Term expires October, 1961:
THOMAS S. FRATAR
EARL F. O'BRIEN
DANIEL B. VENTRES
CHARLES W. BRITZIUS
WAYNE G. O'HARRA
FRED H. RHODES, JR.
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FRED H. RHODES, JR.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 83 (1957)

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6), 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

VOLUME 84 (1958)

JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 (HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 (ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 (SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c].

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858 (SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

c. Discussion of several papers, grouped by divisions.